JOURNAL
OF THE
AMERICAN WATER WORKS ASSOCIATION

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Vol. 33 January, 1941 No. 1

# Trends in Coefficients of Large Pressure Pipes

*By Charles H. Capen*

## Introduction and Summary

EVER since the days of the old Greek and Roman aqueducts, and possibly even before that, engineers have sought to determine the laws of pipe line flow in order to calibrate or calculate accurately the discharge of a given conduit or waterway. The open channels of the past, subject as they were to changes caused by nature, have in a large measure given way to closed conduits. At least this is true insofar as water supply and water power developments are concerned.

For many centuries the hydraulic art of the ancients seems to have been either forgotten or little used, but the extensive studies of the Italian and French hydraulicians of the sixteenth to nineteenth centuries instituted a new conception of the problem. Today, there are so many empirical formulas for determining pipe flow conditions that many engineers are at a loss to pick and choose among them. This paper is written to help those who have such problems.

Possibly few better proving grounds for such conclusions exist than are afforded among the numerous large water pipe lines in northeastern New Jersey. Here are found many innovations in pipe construction, and around these have arisen furious controversies,

A contribution by Charles H. Capen, Engineer, North Jersey Water Supply Commission, Wanaque, N. J.

suits, claims and counter-claims. In each case it has been necessary to establish some definite policy with respect to design or operation. Anyone who has had close contact with such a situation cannot help being impressed by the variations in answers that are given by reputable engineers.

A close association with this problem has led to a number of deductions which are so at variance with commonly accepted tenets that it was decided to set down certain facts for the benefit of those who may wish to study them. The result is a record, given in as much detail as practicable, of the characteristics of the pipe lines studied.

In connection with this study, past and current literature has been reviewed. While references quoted are rather comprehensive, much pertinent material has been omitted wherever it appeared to be a repetition of other facts presented. The references given in the bibliography constitute a hydraulic library which should suffice for all but the most fastidious students. For the benefit of those who may desire further information, the collective references given in the books listed will complete an entirely comprehensive library.

Briefly stated, the import of this paper is that improvements in pipe line construction have enabled water supply engineers to specify such requirements that "smooth bore" conditions are almost certain to result. This is equivalent to saying that most pipes now installed resemble "new pipe" conditions of a few years ago and formulas or coefficients must be modified correspondingly.

Basically the formula recommended as a result of this study is $V = F\ D^{0.625}\ H^{0.5}$, which is merely a simplified method of writing a formula advocated in the past by Hazen, Scobey and, in approximate form, by Fteley. In the cases cited, however, the formula was proposed largely for new or smooth pipes. It remained for Mills to advocate the use and to dèmonstrate the mathematical soundness of the expression, from which the simplified form is derived, as a general equation for all types of pipe problems. Because of this, and because of the fact that his experiments antedated those of the other authors mentioned, credit has been given to him by calling it the Mills equation and by use of the letter $m$ as a subscript to $F$.

In the last sections of this paper in which the selection of a formula and conclusions are discussed, the digest of this entire study is given. For those who may wish to obtain quickly the basic ideas here set forth, the reading of those two sections may suffice.

## Need to Modernize Formulas

Usually a young student in hydraulics receives his first introduction to the problem of pipe line friction losses by way of the Bernoulli theorem. Later there follows a study of practical formulas intended to simplify the application of the theory. Student opinion seems to indicate that the second part of the instruction often has the opposite of the desired effect. The usual result is that each individual selects one particular formula, diagram, or table, from which he can abstract a reasonable answer without too much effort. More often than not the choice of formula or table is based largely on the recommendation or teaching of a particular professor.

In the writer's case, the old standby, upon which emphasis was placed, was the Chezy formula, particularly as used in connection with Kutter's $n$ and suitable diagrams. The impression of validity of this procedure received added reality when it was not only recommended but practically insisted upon by some older engineers who directed certain studies in which the writer took part.

Publications by many contemporary writers began to cast some doubt on the inflexible correctness of applying some of the older ideas regarding pipe lines. Among these were the earlier works by Scobey and the authoritative book by Mills (1). The latter expressed and demonstrated the mathematical proof of certain theories that had apparently originated with him but had been previously published by Merriman (2). The monumental work of Scobey (3) in 1930 was of still further interest.

In 1935, the New England Water Works Association Committee on Pipe Line Friction Coefficients (4) submitted a comprehensive report which, fundamentally, based all its findings on the Williams-Hazen formula. The writer has likewise based many computations on this formula, not only because it permitted ease of comparison with other results, but because the use of tables, supplemented by a Williams-Hazen slide rule, facilitated the work. In spite of this facility, however, it was desired to employ many logarithmic calculations and the use of the Williams-Hazen formula, it must be admitted, is not conducive to ease of operation. To add to the belief in the general use of the formula, Scobey (3, p. 18) says: "Engineers not wishing to adopt the new formula offered in this bulletin, although it conforms to the experimental data, should not digress farther than to use the Williams-Hazen formula, which conforms

with reasonable closeness to that of the writer for usual sizes and velocities."

Having been directly associated with a lengthy and costly argument regarding pipe carrying capacity in the case of the Wanaque aqueduct, the writer has had cause to study the merits of different viewpoints on coefficients. Recent tentative proposals for new water projects in this same area call for a review of the fundamental friction laws before commitments as to future design can be made. Preliminary studies of this kind are particularly necessary if delays of the type mentioned are to be avoided.

Because of certain conflicting opinions, a review of available literature did not alone suffice to form a foundation on which a reliable stand could be predicated. It seemed rather that a comprehensive study of existing pipe lines in the area in question could best supply the information desired. With this in mind, a comparison of several recent tests of friction losses that had been made on some of the large conduits now in use was made. In addition, the literature on the subject was reviewed insofar as possible, with particular reference to tests made on these same pipe lines. Earlier results have, in turn, been compared with recent findings and consideration has been given to all of these toward developing a background on which to predict future trends.

## Pipe Lines in Northern New Jersey

Some of the pioneer work in laying large pipe lines for water supply in the United States was carried out in this area. The first of these was laid in 1892. Since then, many others have been added, and some of the earlier lines have been reconditioned. Tests under these varied circumstances permit useful comparisons of a type that is seldom available.

Pertinent facts regarding these lines, particularly those for which records are available, are given in the subsequent discussion.

## Pequannock Pipe Line No. 1—City of Newark

This was the first of the large pipe lines installed in the region. It was laid by the East Jersey Water Company (under contract with Newark) and was known as their conduit No. 1. The line had a nominal diameter of 48 in., was about 21 mi. long, and was placed in use in April, 1892. Each section of pipe was a cylinder, alternate sections being inner pipes, and the others, outer pipes. This pro-

duced an "in-and-out" pipe, the effective diameter of which was not exactly 48 in., as will be described later. The sections were each 7 ft. long and the laying length, slightly less. The upstream 5 mi. of the line was made of $\frac{1}{4}$-inch plate, while below this the thickness was $\frac{1}{4}$-inch, $\frac{5}{16}$-inch, and $\frac{3}{8}$-inch. Longitudinal joints had two rows, and girth joints, single rows, of button-head rivets. Cylinders were dipped in asphalt in vats. Four lengths were joined together on the bank and then lowered into the trench, thus making every fourth joint a trench operation.

The lengths and thicknesses as given on the profile were as follows: $\frac{1}{4}$-inch, 59,821 ft.; $\frac{5}{16}$-inch, 17,765 ft.; and $\frac{3}{8}$-inch, 34,253—totaling 111,839. (The line was extended 65 ft. in 1896.)

Originally the line had nine valves, all 48 in. in diameter. In 1902, another was inserted at Great Notch, at the point of take-off to the then new Cedar Grove Reservoir.

Horizontal curvature amounted to about 790°, of which 28 per cent was in 5° curves, 50 per cent in 10° curves, and the balance of 22 per cent consisted largely of flat $2\frac{1}{2}$° and sharp 20° curves.

In considering the characteristics of this pipe line, a feature that seems even more important than the horizontal curvature is the amount of vertical rise and fall. The invert elevation at Macopin Intake is 569.2, while at Pompton Notch it is 517.5, a drop of 51.7 ft. In this same stretch, the pipe actually travels through approximately 550 ft. of vertical drop and 500 ft. of rise. These rises and falls involve sixteen grades of more than 10 per cent (omitting one short section that has since been relocated). Seven of these sixteen are greater than 20 per cent, the greatest being 32.9 per cent. Adding the total drop and total rise gives 1,050 ft. of vertical change in elevation or 37.3 ft. per thousand feet of pipe (the horizontal distance being 28,311 ft.). This is shown, together with similar data for the other parts of the pipe line, in a comparison given in Table 1.

Negotiations for the complicated legal contract for and the construction of this line covered several years. It seems probable that, initially, the engineers planning it contemplated a largely grade aqueduct. This belief is strengthened by the fact that in the first section between Macopin Intake and Pompton Notch (which is a high narrow gorge), a distance of 28,311 ft., the pipe line appears to have been laid out as close to the hydraulic gradient as possible. A study of the topography indicates that in many places the route could readily have been changed to avoid approaching the hydraulic

gradient, and that this could have been done at an actual saving in cost. From a standpoint of practical operation, a careful shunning of the high points along this section of the pipe line, in the early purchase of rights of way, together with a slightly deeper cut through

TABLE 1

*Pequannock Conduit No. 1 Construction Data*

| ITEM | UNIT | MACOPIN INTAKE TO POMPTON NOTCH | POMPTON NOTCH TO GREAT NOTCH CROSS-OVER | GREAT NOTCH TO BELLEVILLE RESERVOIR | ENTIRE LINE |
|---|---|---|---|---|---|
| Length (in plan) | Feet | 28,311 | 50,424 | 33,147 | 111,882 |
| Total horizontal curvature | Degrees | 423* | 128 | 237 | 788 |
| Horizontal curvature per thousand feet | " | 14.9* | 2.5 | 7.2 | 7.0 |
| Nominal drop in elevation | Feet | 51.7 | 237.6 | 130.0 | 419.2 |
| Total drop in elevation | " | 551 | 865 | 410 | 1,826 |
| " rise " " | " | 500 | 627 | 280 | 1,407 |
| " change" " | " | 1,051 | 1,492 | 690 | 3,233 |
| " " " " per thousand feet | " | 37.3 | 29.7 | 20.8 | 28.8 |
| Vertical changes in grade | Each | 96 | 175 | 125 | 396 |
| " " " " per thousand feet | " | 3.40 | 3.48 | 3.79 | 3.54 |
| Air valve chambers | " | 20 | 28 | 23 | 71 |
| " " " per thousand feet | " | 0.71 | 0.56 | 0.69 | 0.64 |
| Blow-offs | " | 14 | 30 | 19 | 63 |
| " " per thousand feet | " | 0.50 | 0.59 | 0.57 | 0.56 |
| Manholes | " | 24 | 43 | 28 | 95 |
| " per thousand feet | " | 0.85 | 0.85 | 0.84 | 0.85 |

* After relocation of a short stretch of line in 1934, total became 616° or 21.7° per thousand feet.

Pompton Notch, would have removed many of the engineering difficulties that have occurred during the nearly half century of operation.

To indicate the necessity for decisive action by the East Jersey Water Company, it may be pointed out that the Company's contract with the City of Newark required that the system should be

capable of supplying 50,000,000 gallons per day (50 m.g.d.). A clause in the contract gave the City the option of purchase by 1900. Herschel (5, p. 5), then Chief Engineer of the Company, indicated that the two guiding motives in the selection of pipe material were: (A) the time available for construction under the contract, and (B) the pressure that would prevail. He said that no cast-iron or masonry pipes could be built in the two seasons allotted and that no cast-iron pipe of this size, under a 340-foot head, had been built. The bold step of selecting steel for such conditions was without precedent in the East. Apparently the engineers felt reasonably sure of their estimates, as the hydraulic gradient was drawn on the profile of the pipe line in a fixed position. This gradient just touched the top of the pipe along sections where the pipe was laid at estimated grade aqueduct conditions.

Tests made in 1892, during the first few months of operation, evidently revealed inability of the pipe to meet the requirements of the contract. Subsequent tests in 1893, 1894 and 1896 confirmed this, while at the same time showing an alarming decrease in capacity as the purchase date drew closer. Herschel (5) wrote his book on the subject partly to vindicate his own actions. He accepted full responsibility for the design and said: (p. 5) "No allowance was made in the computation for deterioration of carrying capacity by the formation of tubercles. This was omitted because it was then supposed that steel pipes would not deteriorate in this way, like cast-iron pipes, or, as stated in Hamilton Smith's *Hydraulics*, 'would remain free from rust and tubercles.' "

Herschel, using a table from Smith's book (p. 271) computed a diameter of $47\frac{1}{4}$ in. and a slope of 10.56 ft. per mile for the line. Emil Kuichling who had made tests, over a period of years, on the 36-inch steel main in Rochester was called upon to check Herschel's calculations. Using certain of his own observations as a guide, and employing the Lampe formula in the form $S = 0.00039211 \frac{V^{1.802}}{D^{1.25}}$, Kuichling obtained a required diameter of 47 in. with a slope of 11.8 ft. per mile. This was substantially the same as Herschel's estimate. Later, in his book, Herschel wrote a chapter, entitled "The Rochester Crime," in which he stated his belief that only the more favorable of the Rochester tests had been considered and that these did not represent a true picture of the condition of the steel pipe line.

By 1896, the maximum capacity of line No. 1 was 35 m.g.d., from which Herschel estimated the original capacity to have been 43 m.g.d. (instead of the 50 m.g.d. required). As a result, the Company found it necessary to parallel the first conduit with another known as conduit No. 2.

### Pequannock Pipe Line No. 2

Mills (1, p. 214) says: "Pipe laying commenced in March, 1896. It was completed, and the conduit put into use, to Pompton Notch, on May 30, 1896, and completed to Belleville and put into use for its entire length on Sept. 30, 1896."

Calculations and observations had shown that the upper 5 mi. of the first conduit formed the bottle neck. Consequently the upper section of the second line was made 48 in. in diameter in the critical stretch above Pompton Notch but was reduced to 42 in. in diameter below that point. It was constructed with taper joints instead of the in-and-out cylinder joints used in the previous case. The section between Macopin and Pompton Notch resembled the No. 1 line in all other respects except for the fact that it had no line valve until it reached the Notch, whereas the No. 1 line had an intermediate valve. In the whole length of the No. 2 line there was only one other valve, a 42-inch one, at Great Notch, which was inserted in 1902 when Newark constructed the Cedar Grove Reservoir.

A summary of the thicknesses and lengths of the No. 2 line is given below:

| | |
|---|---|
| Above Pompton Notch (all 48-inch): | |
| $\frac{1}{4}$ in. thick | 27,858 ft. |
| $\frac{3}{8}$ in. thick | 428 ft. |
| | 28,286 ft. |
| Below Pompton Notch (42-inch, except for river crossings): | |
| $\frac{1}{4}$ in. thick | 45,168 ft. |
| $\frac{5}{16}$ in. thick | 37,209 ft. |
| | 82,377 ft. |
| 48-inch, $\frac{3}{8}$ in. thick | 971 ft. |
| | (river and canal crossings) |
| Reducer 48 x 42 in., $\frac{1}{4}$ in. thick | 6.8 ft. |
| | (one; at Pompton Notch) |
| Reducer 48 x 42 in., $\frac{3}{8}$ in. thick | 47 ft. |
| | (six; at each end of crossings of Pompton River, Passaic River and Morris Canal) |

Recapitulation:

| | |
|---|---|
| 48-inch | 29,257 ft. |
| 42-inch | 82,377 ft. |
| Reducers | 54 ft. |
| Total | 111,688 ft. |

The horizontal curvature (except for two river crossings) was as follows:

| DEGREE OF CURVE | APPROXIMATE RADIUS OF CURVATURE | CENTRAL ANGLE (TOTALS) |
|---|---|---|
| | *ft.* | |
| 2°30′ | 2,500 | 87°11′ |
| 5° | 1,165 | 217°03′ |
| 7°30′ | 760 | 48°49′ |
| 10° | 580 | 393°58′ |
| Sharp bend | 74 | 41°30′ |
| | | 788°31′ |

Each of two river crossings had two 20° curves with central angles of 16°04′ each or a total of 64°16′. Therefore, the total horizontal angle of curvature on this line was 852°47′.

Above Pompton Notch the original horizontal angles totaled 423°23′ or about the same as for line No. 1. In 1934, a highway relocation required moving a short stretch of pipe, introducing additional sharp curves in each line. Since the pipe relocation began and ended on existing curves, it removed about 11° of central angle from the original lines and added the following to the No. 2 line (the No. 1 line being quite similar):

| RADIUS OF CURVE | CENTRAL ANGLE |
|---|---|
| *ft.* | |
| 29.00 | 61°45′ |
| 50.48 | 77°20′ |
| 286.10 | 5°28′ |
| 52.0 | 59°26′ |
| | 203°59′ or about 204° |

The net increase in horizontal curvature was 204° minus 11°, or 193°. The length was increased 72.1 ft., but the vertical rise and fall were not materially changed.

As noted at the bottom of Table 1, this increased the curvature

per thousand feet of length from 14.9° to 21.7°. The relocated section was about 600 ft. long and was laid with pipe $\frac{3}{8}$ in. in thickness, in lengths of 30 ft., with button-head riveted longitudinal and girth joints. This new pipe was lined with bitumastic enamel.

Between Pompton Notch and Great Notch the No. 2 line had 194° of curvature or 3.8° per thousand feet of length. In most other respects, except diameter, it was very similar to the No. 1 line.

*Tests on Pequannock Pipes Prior to 1930.*

Tests on the No. 1 line were made by Herschel immediately after it was placed in operation in April, 1892, and at various periods thereafter. The tests were divided between various portions of the line and covered a fairly wide range of velocities. In 1896, tests were made on both lines and all of the results of the four years have been reported by Herschel (5).

There is no further record of tests by Herschel and, since the City of Newark took over the system in 1900, it seems likely that the laying of conduit No. 2 obviated the necessity of close observation of carrying capacity and coefficients. In 1907, Merriman made a series of observations which have been very useful in depicting the progressive changes in these lines.

In 1923, the New York Board of Water Supply became interested in obtaining up-to-date information on these lines and probably Merriman had a part in the investigation that resulted. Records of this investigation on file with the City of Newark, do not, at first glance, reveal much useful information; but by taking average results over most of the period of the tests, fairly reasonable results may be obtained. This set of tests must not be regarded with as much weight as most of the others, largely because the Venturi meter readings are not consistent.

Because of peak drafts, and delays in completing cooperative arrangements necessary in the construction of the joint Wanaque project, it became necessary for Newark (1925) to lay a 48-inch wood stave line, about 3 mi. long, from Charlotteburg Pond to connect with the No. 1 line at Macopin. This added about 117 ft. of head to the line and increased the carrying capacity materially. The wood stave line was to have served for a short period of time only, but actually it was used for several years, although subject to many leaks.

*Relining; and Subsequent Tests.*

With the completion of the Wanaque project in 1930, the wood stave line became less important and, in fact, deterioration made it unreliable. As a result of this, capacity of the two conduits again

Tuberculation in Pequannock 48-Inch Line Before Cleaning—After 39 Years' Use; Coefficient 70 (Williams-Hazen Formula)

rose in importance. Additional tests were made in 1933, and shortly thereafter it was decided to recondition the No. 2 line.

A very interesting and comprehensive description of the work of cleaning and relining the line is given by Banks and Inglee (6). They say, in part, that the first reconditioning was undertaken in 1933 and consisted of removing tubercles and other material by

scrapers and brushes, cleaning with rags, applying a cold priming coat, and finally applying a hot (425° F.) bituminous enamel $\frac{1}{8}$ in. thick. This process was followed, with interruptions, on the lower part of the line, and by the time the article was written, about 8,000 ft. had been reconditioned in this manner.

In the latter part of 1935, prior to awarding a reconditioning contract, tests were again made on the No. 2 line. A contract was awarded to the T. A. Gillespie Company (which had laid the lines originally for the East Jersey Water Company) to clean and reline, with cement mortar, the portion of the No. 2 line between Macopin Intake and Pompton Notch. Cleaning of the line was sublet to the National Water Main Cleaning Company. Banks and Inglee (6) have described this process in great detail and it need not be further elaborated on at this point. Of particular interest is the fact that various thicknesses of cement mortar, depending on the condition of the plate and local hazards, were used. Normal thickness was $\frac{1}{4}$-inch, but certain portions were lined with a $\frac{5}{8}$-inch coating. In a section under a road and one under a stream, a 1-inch thickness was applied. Two stretches were not relined, one being the relocated portion, of about 600 ft. in length, previously described, the other, a portion 273 ft. in length which was so far out of round that it did not lend itself well to the process. Work started in November, 1935, and was completed in May, 1936.

In 1936, one set of observations was made after the line had been cleaned, but before lining. In the latter part of 1936, after the lining was completed, and again in the early part of 1937, the Newark Water Department made further tests. In some of these a unique method was used. A set of points was selected at the ends of the sections representing the approximate limits of the various thicknesses of mortar lining used in the reconditioning. The flow was set at a steady rate and a long column of mercury was carried by car to each point, a reading of pressure was taken quickly, and the observers proceeded to the next station. By traversing the line several times in this manner a reasonable and comprehensive set of results was obtained.

To bring the records up to date, and to correlate all the work with the interconnection studies recently made for the purpose of dovetailing the many existing supplies in the area, the writer and those concerned with the study cooperated with the Newark Water Department in making additional tests. These observations, made

in the early part of 1940, were carried out with carefully calibrated pressure gages except at one location where the presence of the hydraulic gradient near the ground level permitted use of a rubber hose and long glass tube. (Herschel used glass tubes on specially constructed wooden towers.)

The flow in the line had been held steady for some time and observations of flow at 5-minute intervals over a $2\frac{1}{2}$-hour period showed a maximum variation of from 22.6 to 22.8 m.g.d. as taken from the Venturi meter at the upper end of the line. Pressure readings were also taken at 5-minute intervals and in most instances little or no variations occurred. The maximum variation in the water column was 9 in., but the range, during most of the period, was only 3 in. Under such conditions, and considering the great lengths of pipe involved in the main sections, results are considered as reliable as may reasonably be expected.

*Comparison of 48 Years of Tests.*

As far as reliability of the various tests is concerned, it may be said, in general, that many of the observations of short lengths(*i.e.* less than 1 mi.) cannot be regarded as fully representative while those covering sections 2 mi. or more in length may be accorded a reasonable degree of consideration. Sections 5 mi. or more in length may be given definite value in most cases, except where variations in flow or inaccuracies in recording it during the tests were such that obvious discrepancies were encountered. A careful review of the voluminous data available shows conclusively that variations in a water column up to 9 in. (as actually observed on more than one occasion), even under the very best possible conditions, together with other difficulties attendant to the making of observations, are of such an order as to make isolated readings over short distances not wholly dependable. It seems probable that Herschel, Merriman and others must have come to this same conclusion, because the only figures they have released with any degree of assurance are those covering long distances between points of observations.

Because so many of the results given in the past have been expressed in terms of the Williams-Hazen coefficient (designated in this paper as $C_w$), this unit will be used in a large part of the comparison. Later in the paper, the comparative merits of various coefficients will be discussed.

TABLE 2

*Pequannock Conduits—Effective Diameter*

(as given by Mills)

Line No. 1 (1892). Nominal diameter = 48 in. for entire length.
Line No. 2. (1896). Nominal diameter = 48 in. from Macopin to Pompton Notch. Nominal diameter = 42 in. from Pompton Notch to Belleville.

| | MACOPIN INTAKE TO POMPTON NOTCH | POMPTON NOTCH TO GREAT NOTCH | GREAT NOTCH TO BELLEVILLE |
|---|---|---|---|
| Line No. 1: Effective diameter, *ft.* | 3.96 | 3.96 | 3.96 |
| Hydraulic radius, *ft.* | 0.99 | 0.988 and .985 | 0.985 |
| Line No. 2: Effective diameter, *ft.* | 3.96 | 3.50 | 3.50 |
| Hydraulic radius, *ft.* | 0.99 | 0.875 | 0.875 |

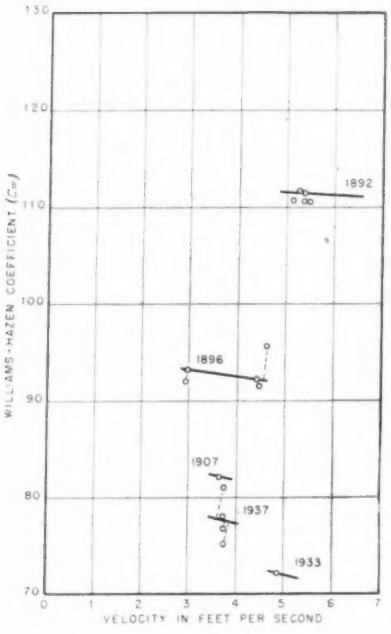



FIG. 1

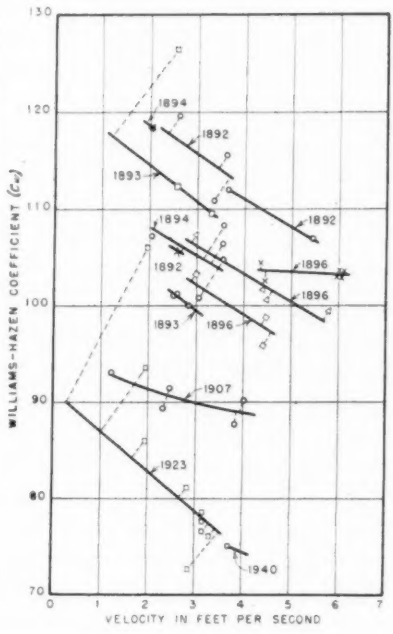



FIG. 2

FIG. 1. Variation of Coefficient; 48-Inch Pequannock Conduit No. 1 Above Pompton Notch

FIG. 2. Variation of Coefficient; 48-inch Pequannock Conduit No. 1 Below Pompton Notch

It should be noted that Herschel (5) made allowances for the character of the pipe, with respect to its nominal diameter, and that his figures all showed estimates of less than the 48-inch nominal value. In the case of the 42-inch main, however, he used the nominal value. A record of these variations is given in Table 2.

Present practice in dealing with these lines is to use the full nominal diameters of 48 in. and 42 in. ($R$, hydraulic radius, equaling 1.0 and 0.875 respectively), particularly since variations in the thickness of pipe coatings or linings outweigh the slight difference between nominal and actual diameter.

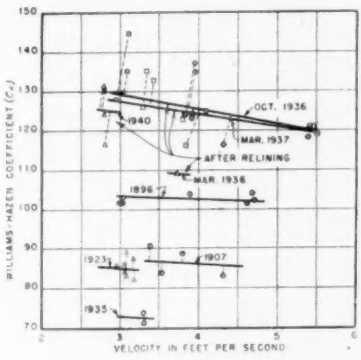



FIG. 3

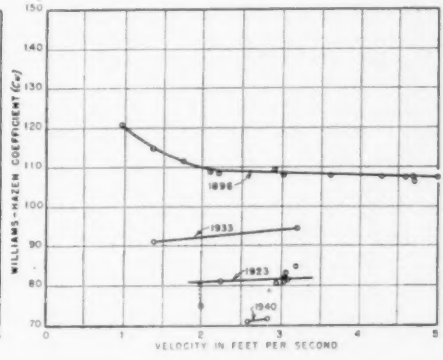



FIG. 4

FIG. 3. Variation of Coefficient; 48-Inch Pequannock Conduit No. 2 Above Pompton Notch

FIG. 4. Variation of Coefficient; 42-Inch Pequannock Conduit No. 2 Below Pompton Notch

Early tests on these conduits have all been recorded in the various publications heretofore mentioned and need not be repeated here. Scobey (3) has not only collected most of these early data but has computed and recorded the comparison of coefficients found by using various formulas. The writer has checked all of Scobey's figures with respect to the Williams-Hazen formula and agrees with them closely. The checking was done both by logarithmic computation and by use of the the Williams-Hazen hydraulic slide rule.

While no tabular recording of these early results is to be repeated, a graphic comparison of all figures, both early and recent, is considered useful. After some study it appeared that graphs showing

the relation of $C_w$ to $V$ (velocity) for the various years would best demonstrate the points stressed in this paper.

Accordingly, for No. 1 conduit, there is given in Fig. 1, the relation of $C_w$ to velocity, during the years of operation, for the stretch above Pompton Notch. Figure 2 shows the same relation for the portion below Pompton Notch. Figures 3 and 4 are the same respective conditions for conduit No. 2.

The decrease of $C_w$ with age is clearly shown, and the decrease of $C_w$ with increasing velocity is so consistent that immediate doubt is cast on the value of the Williams-Hazen formula. Both of these points are further commented on in later paragraphs.

TABLE 3

*Herschel's Experimental Data on Conduit No. 1 and Extension*

| VELOCITY (FT. PER SEC.) | $C$ NEW | $C$ ABOVE POMPTON NOTCH, 4 YR. OLD | $C$ BELOW POMPTON NOTCH, 4 YR. OLD | $C$ 36-INCH SECTION, NEW* |
|---|---|---|---|---|
| 1.0 | 101.2 | 78.0 | 97.2 | 86.0 |
| 1.5 | 105.4 | 84.6 | 100.8 | 90.8 |
| 2.0 | 108.8 | 89.6 | 103.3 | 95.2 |
| 2.5 | 111.2 | 92.4 | 104.9 | 99.4 |
| 3.0 | 112.8 | 93.0 | 105.3 | 103.3 |
| 3.5 | 113.4 | 93.2 | 104.8 | 107.0 |
| 4.0 | 113.2 | 94.0 | 104.0 | 110.6 |
| 4.5 | 112.4 | 94.2 | 103.7 | 114.0 |
| 5.0 | 112.0 | 94.4 | 103.7 | 117.2 |
| 5.5 | 111.7 | 94.7 | 103.7 | 120.4 |
| 6.0 | 111.6 | 94.9 | 103.7 | 123.6 |

* When 36-inch pipe was 4 yr. old, $C = 106.3$ at velocity of 4.93.

It is pertinent to note, at this point, that Herschel (5, p. 52) summarized his own results by tabulating values of the Chezy coefficient ($C$) which he deduced from the experimental data on conduit No. 1 and its extension to South Orange Avenue Reservoir. These results are shown in Table 3.

*Effect of Age, Location and Relining on Carrying Capacity.*

The deterioration in the pipes, and its relation to distance from the intake, may be more fully appreciated by a study of Figs. 5 and 6. These show the value of $C_w$ as one ordinate and the distance from Macopin as the other. Mills (1, p. 210) reports that Merriman, after having observed the interior of the No. 2 pipe on May 21, 1907, made the following statement.

"At Sta. 251 + 27 (near Pompton Notch), the coating is very rough and quite brittle. It shows a number of blisters, and the tubercles are not in excess of 25 per square foot, though some sheets are decidedly worse than this. The deepest pitting found was about $\frac{1}{16}$ inch. The pipe here is always full of water.

"Below the Venturi meter at Sta. 13 + 00, the coating is as rough as at Sta. 251 + 27. There are also more tubercles—I estimate about 40 per square foot. The number of blisters is about the same as where previously seen and there is no rusting under them. The deepest pitting found was about $\frac{1}{16}$ inch. The most marked point of difference between the condition of this pipe here and where observed elsewhere is that there is considerably more organic deposit. At Sta. 825 + 69 there was practically none;

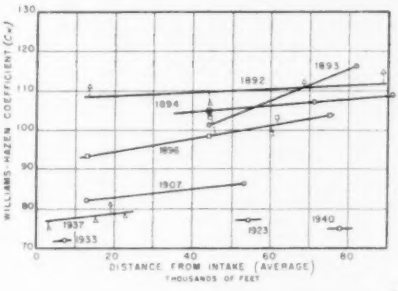



FIG. 5

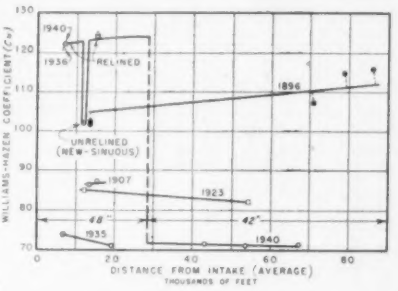



FIG. 6

FIG. 5. Relation of $C_w$ to Distance From Intake; Pequannock Conduit No. 1(48-Inch)

FIG. 6. Relation of $C_w$ to Distance From Intake; Pequannock Conduit No. 2 (48-Inch and 42-Inch)

at Sta. 251 + 27 there was about as much as could be represented by the thickness of a fairly heavy paper, while at 13 + 00 its thickness would average about $\frac{1}{16}$ inch."

Merriman also said that the body of the Venturi meter was of wood and was quite rough, due to the dissolution of the wood. The heaviest organic growths were at the meter, the deposit being $\frac{1}{8}$ in. thick on the brass throat piece, while above and below, it was in places as much as $\frac{3}{4}$ in. thick.

Apparently some reason for the greater losses near the intake is established. A study of Figs. 5 and 6, however, leads to the belief that the coefficient has tended to become more uniform throughout the entire length of the aqueduct as the age has increased. This would indicate that the coefficient has gradually approached a lower

TABLE 4

*Tests on 48-inch Pequannock Conduit No. 1*

From 1923 to 1940

Riveted pipe—7 foot lengths. Nominal D = 4′-0″ $D^{1.25} = 5.657$ $D^{0.625} = 2.378$

| TEST NO. | DATE | AGE OF PIPE | OBSERVATION RATING | LOCATION OF TEST SECTION STATION TO STATION | | $L$ LENGTH OF TEST SECTION | AVERAGE DISTANCE FROM INTAKE | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = M'$ | TEMPERATURE OF WATER | $C_w$ WILLIAMS-HAZEN | $F_m$ MILLS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | Total | Per thousand feet $H$ | | | | |
| | | *years* | | | | *feet* | *feet* | *m.g.d.* | *feet per second* | | *feet* | *feet* | | °C. | | |
| P-1 | Nov. 8, 1923 | 31.5 | C | 298 + 35 | 781 + 11 | 48,276 | 53,973 | 26.7 | 3.29 | 10.82 | 85.91 | 1.78 | 0.164 | | 76 | 1.04 |
| | | | C | | | | | 25.7 | 3.16 | 9.99 | 76.23 | 1.58 | .158 | | 78 | 1.06 |
| | | | C | | | | | 25.7 | 3.16 | 9.99 | 79.73 | 1.655 | .166 | | 76.5 | 1.04 |
| P-2 | Nov. 9, 1923 | 31.5 | C | 298 + 35 | 781 + 11 | 48,276 | 53,973 | 23.1 | 2.84 | 8.07 | 60.09 | 1.245 | .154 | | 81 | 1.07 |
| | | | C | | | | | 23.1 | 2.84 | 8.07 | 71.64 | 1.485 | .184 | | 72.5 | 0.98 |
| | | | D | | | | | 15.75* | 1.94 | 3.76 | 25.44 | 0.527 | .140 | | 86 | 1.13 |
| | | | D | | | | | 15.75* | 1.94 | 3.76 | 21.97 | 0.457 | .122 | | 93.5 | 1.21 |
| P-3 | Nov. 13, 1923 | 31.5 | C | 298 + 35 | 781 + 11 | 48,276 | 53,973 | 25.7 | 3.16 | 9.99 | 77.38 | 1.60 | .160 | 15 | 77.5 | 1.05 |
| | | | C | | | | | 25.7 | 3.16 | 9.99 | 76.23 | 1.575 | .158 | | 78.5 | 1.06 |
| | | | D | | | | | 15.7* | 1.93 | 3.72 | 17.35 | 0.36 | .097 | | 106 | 1.35 |
| P-4 | Apr. 17, 1933 | 41 | B | 16 + 75 | 122 + 68 | 10,593 | 6,966 | 40.0 | 4.93 | 24.30 | 44.59 | 4.20 | .173 | 9 | 72 | 1.01 |
| P-5 | May 12, 1937 | 45.1 | B | 16 + 75 | 48 + 26 | 3,151 | 3,250 | 30.4 | 3.74 | 13.99 | 7.25 | 2.30 | .165 | 14 | 75 | 1.04 |
| P-6 | May 12, 1937 | 45.1 | B | 143 + 98 | 154 + 06 | 1,008 | 14,902 | 30.4 | 3.74 | 13.99 | 2.26 | 2.24 | .160 | 14 | 77 | 1.05 |
| P-7 | May 12, 1937 | 45.1 | B | 154 + 06 | 194 + 03 | 3,997 | 17,404 | 30.4 | 3.74 | 13.99 | 8.09 | 2.02 | .144 | 14 | 81 | 1.11 |
| P-8 | May 12, 1937 | 45.1 | B | 194 + 03 | 225 + 23 | 3,120 | 20,963 | 30.4 | 3.74 | 13.99 | 6.72 | 2.15 | .154 | 14 | 78 | 1.07 |
| P-9 | Apr. 25, 1940 | 48 | A | 575 + 38 | 781 + 11 | 20,573 | 67,824 | 30.0 | 3.69 | 13.62 | 45.73 | 2.23 | .164 | 6† | 75 | 1.03 |

* $Q$ probably incorrect.

† Temperatures given for April 25, 1940 are in conduit; all other temperatures were taken in city and are probably slightly higher than in conduit.

limit in all parts of the aqueduct and that the decrease has become progressively less for the same time interval as the age has increased. It should be borne in mind, too, that the horizontal and vertical curves, as well as the number of air valves, blow-offs, and other appurtenances, affect the coefficient in one section as compared with that in another. Reference to Table 1 will show that this effect is greatest near the intake because of the rugged terrain.

To demonstrate further the change with age, Fig. 7 has been prepared to show the coefficients of the two lines during the period of operation. (Figure 7 also shows curves for other pipe lines to be described in later paragraphs.) These curves illustrate clearly

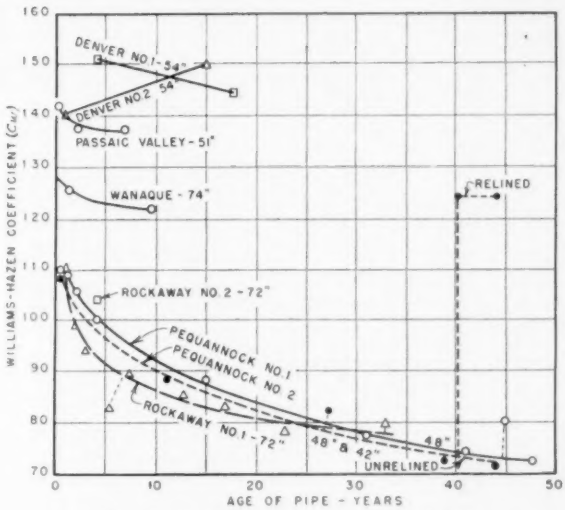



FIG. 7. Relation of $C_w$ to Age of Pipe

that the coefficients have shown a distinct tendency to level off during the last several years. Also there is shown graphically, for the first time (so far as known), the lasting improvement in the coefficient of the upper (48-inch diameter) portion of conduit No. 2, as a result of the cement lining in 1936. It should be noted that in this upper portion, as mentioned previously, two short stretches were not relined. Because of this, the comparative coefficient should be slightly higher than any result found in practice.

The individual results for some of the later tests of conduits No. 1 and 2, not previously published, are given respectively in Tables 4 and 5. These tables will permit application of the figures to any desired formula:

TABLE 5

PART 1. *Tests on 48-Inch Portion of Pequannock Conduit No. 2—From 1923 to 1940*

Riveted pipe: 7-foot lengths; Nominal $D$ = 4.0 ft.; $D^{1.25}$ = 5.657; $D^{0.625}$ = 2.378; Relined early in 1936

| TEST NO. | DATE | AGE OF PIPE | OBSERVATION RATING No. | LOCATION OF TEST SECTION STATION TO STATION | | $L$ LENGTH OF TEST SECTION | AVERAGE DISTANCE FROM INTAKE | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = \frac{1}{M'}$ | TEMPERTURE OF WATER | $C_W$ WILLIAMS-HAZEN | $F_m$ MILLS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | Total | Per thousand feet $H$ | | | | |
| | | *years* | | | | *feet* | *feet* | *m.g.d.* | *feet per second* | | *feet* | *feet* | | °C. | | |
| P-10 | Nov. 8, 1923 | 27.4 | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 24.6 | 3.03 | 9.18 | 26.64 | 1.275 | 0.139 | | 84 | 1.13 |
| | | | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 24.6 | 3.03 | 9.18 | 25.48 | 1.220 | .133 | | 86 | 1.16 |
| P-11 | Nov. 9, 1923 | 27.4 | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 24.8 | 3.05 | 9.30 | 24.33 | 1.161 | .125 | | 89 | 1.19 |
| | | | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 25.7 | 3.16 | 9.99 | 30.10 | 1.44 | .144 | | 82 | 1.11 |
| | | | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 25.7 | 3.16 | 9.99 | 26.64 | 1.28 | .128 | | 87.5 | 1.18 |
| P-12 | Nov. 13, 1923 | 27.4 | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 24.8 | 3.05 | 9.30 | 27.79 | 1.33 | .143 | | 83 | 1.12 |
| | | | C | 16 + 45 | 225 + 31 | 20,886 | 12,088 | 23.75 | 2.92 | 8.53 | 24.33 | 1.162 | .136 | | 85.5 | 1.14 |
| P-13* | Aug., 1935 | 39.2 | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 26.8 | 3.30 | 10.89 | 19.25 | 1.92 | .176 | 24 | 73.5 | 1.00 |
| P-14† | Mar., 1936 | 39.8 | B | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 30.2 | 3.72 | 13.84 | 11.45 | 1.14 | .105 | 6 | 109 | 1.47 |
| P-15 | Jun., 1936 | 40.0 | C | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 33.0‡ | 4.06 | 16.48 | 11.32 | 1.125 | .0681 | 20 | 120.5 | 1.61 |
| P-16 | Oct., 1936 | 40.3 | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 35.0 | 4.31 | 18.58 | 13.53 | 1.345 | .0723 | 18 | 116 | 1.57 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 35.0 | 4.31 | 18.58 | 12.78 | 1.270 | .0683 | 18 | 120 | 1.61 |

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| P-16 | Oct., 1936 | 40.3 | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 35.0 | 4.31 | 18.58 | 13.53 | 1.345 | .0723 | 18 | 116 | 1.57 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 35.0 | 4.31 | 18.58 | 12.78 | 1.270 | .0682 | 18 | 120 | 1.61 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 25.0 | 3.08 | 9.49 | 5.45 | 0.542 | .0571 | 18 | 135 | 1.76 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 43.6 | 5.37 | 28.84 | 19.67 | 1.955 | .0678 | 18 | 118 | 1.62 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 31.7 | 3.90 | 15.21 | 10.07 | 1.00 | .0657 | 18 | 123 | 1.64 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 31.7 | 3.90 | 15.21 | 9.92 | 0.987 | .0649 | 18 | 124 | 1.65 |
| P-17 | Apr., 1937 | 40.8 | B | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 23.7 | 2.92 | 8.53 | 5.47 | 0.544 | .0638 | 8 | 128 | 1.67 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 44.2 | 5.44 | 29.59 | 19.87 | 1.975 | .0667 | 8 | 119 | 1.63 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 31.0 | 3.82 | 14.59 | 10.75 | 1.07 | .0732 | 8 | 116 | 1.56 |
| | | | A | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 30.7 | 3.78 | 14.29 | 9.57 | 0.953 | .0667 | 8 | 123 | 1.63 |
| P-18 | Apr., 25, 1940 | 43.8 | B | 16 + 45 | 116 + 98 | 10,053 | 6,671 | 22.7 | 2.80 | 7.84 | 6.07 | 0.604 | .0771 | 8¶ | 116 | 1.52 |
| P-19 | Apr., 25, 1940 | 43.8 | A | 16 + 45 | 283 + 08 | 26,735 | 15,007 | 22.7 | 2.80 | 7.84 | 14.35 | 0.538 | .0687 | 8¶ | 124 | 1.61 |
| P-20* | Aug., 1935 | 39.2 | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 26.8 | 3.30 | 10.89 | 28.43 | 2.08 | .191 | 24 | 71 | 0.96 |
| P-21† | Mar., 1936 | 39.8 | B | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 30.2 | 3.72 | 13.84 | 12.35 | 0.908 | .0657 | 6 | 124 | 1.65 |
| P-22 | Jun., 1936 | 40.0 | C | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 33.0‡ | 4.06 | 16.48 | 14.06 | 1.032 | .0625 | 20 | 126.5 | 1.68 |
| P-23 | Oct., 1936 | 40.3 | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 25.0 | 3.08 | 9.49 | 6.48 | 0.476 | .0502 | 18 | 145 | 1.88 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 43.6 | 5.37 | 28.84 | 25.65 | 1.88 | .0651 | 18 | 121 | 1.65 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 31.7 | 3.90 | 15.21 | 11.68 | 0.856 | .0563 | 18 | 135 | 1.78 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 31.7 | 3.90 | 15.21 | 11.39 | 0.835 | .0549 | 18 | 136 | 1.80 |

* Before lining.

† After lining.

‡ $Q$ varied from 32.6 to 33.4.

§ $Q$ probably incorrect.

¶ Temperatures given for April 25, 1940 are in conduit; all other temperatures were taken in City and are probably slightly higher than in conduit.

TABLE 5—*Concluded*

PART 1. *Tests on 48-Inch Portion of Pequannock Conduit No. 2—From 1923 to 1940—Continued*

| TEST NO. | DATE | AGE OF PIPE | OBSERVATION RATING | LOCATION OF TEST SECTION STATION TO STATION | | $L$ LENGTH OF TEST SECTION | AVERAGE DISTANCE FROM INTAKE | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = \frac{1}{M'}$ | TEMPERATURE OF WATER | $C_W$ WILLIAMS-HAZEN | $F_m$ MILLS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | Total | Per thousand feet $H$ | | | | |
| | | *years* | | | | *feet* | *feet* | *m.g.d.* | *feet per second* | | *feet* | *feet* | | °*C.* | | |
| P-24 | Apr., 1937 | 40.8 | B | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 23.7 | 2.92 | 8.53 | 7.00 | 0.514 | .0603 | 8 | 133 | 1.72 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 44.2 | 5.44 | 29.59 | 25.95 | 1.905 | .0643 | 8 | 121 | 1.66 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 31.0 | 3.82 | 14.59 | 11.07 | 0.812 | .0558 | 8 | 135 | 1.78 |
| | | | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 30.7 | 3.78 | 14.29 | 12.34 | 0.903 | .0631 | 8 | 126 | 1.68 |
| P-25 | Apr. 25, 1940 | 43.8 | A | 123 + 19 | 259 + 34 | 13,621 | 19,129 | 22.7 | 2.80 | 7.84 | 6.53 | 0.48 | .0612 | 6¶ | 131 | 1.70 |
| P-26 | Apr. 25, 1940 | 43.8 | A | 123 + 19 | 283 + 08 | 15,995 | 20,316 | 22.7 | 2.80 | 7.84 | 7.76 | 0.486 | .0620 | 6¶ | 130 | 1.69 |

PART 2. *Tests on 42-Inch Portion of Pequannock Conduit No. 2—From 1923 to 1940*

Riveted pipe: 7-foot lengths; Nominal $D$ = 3.5 ft.; $D^{1.25}$ = 4.787; $D^{0.625}$ = 2.188

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| P-27 | Apr. 18, 1940 | 43.8 | B | 283 + 08 | 575 + 82 | 29,274 | 42,945 | 20.7 | 3.33 | 11.09 | 71.50 | 2.44 | 0.220 | 7 | 71 | 0.98 |
| P-28 | Apr. 25, 1940 | 43.8 | A | 283 + 08 | 575 + 82 | 29,274 | 42,945 | 22.7 | 3.65 | 13.32 | 84.26 | 2.88 | .216 | 6¶ | 72 | 0.99 |
| P-29 | Sept., 1933 | 37.2 | B | 298 + 24 | 575 + 82 | 27,758 | 43,703 | 26.5 | 4.26 | 18.15 | 63.00 | 2.27 | .125 | 21 | 94 | 1.30 |
| | | | B | | | | | 11.0 | 1.77 | 3.13 | 13.34 | 0.48 | .153 | | 90 | 1.17 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| P-30 | Nov. 8, 1923 | 27.4 | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 24.6 | 3.96 | 15.68 | 128.91 | 2.67 | .170 | | 81 | 1.11 |
| | | | C | | | 48,251 | 53,949 | 24.6 | 3.96 | 15.68 | 127.76 | 2.64 | .168 | | 81.5 | 1.12 |
| P-31 | Nov. 9, 1923 | 27.4 | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 24.8 | 3.99 | 15.92 | 126.60 | 2.62 | .165 | | 81.5 | 1.13 |
| | | | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 18.0 | 2.89 | 8.35 | 71.16 | 1.47 | .176 | | 80.5 | 1.10 |
| | | | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 25.7 | 4.13 | 17.06 | 125.45 | 2.61 | .153 | | 84.5 | 1.17 |
| P-32 | Nov. 13, 1923 | 27.4 | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 24.8 | 3.99 | 15.92 | 128.91 | 2.67 | .168 | | 81 | 1.12 |
| | | | C | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 24.8 | 3.99 | 15.92 | 124.29 | 2.57 | .160 | | 83 | 1.14 |
| | | | | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 23.75 | 3.82 | 14.59 | 121.98 | 2.52 | .173 | | 80.5 | 1.10 |
| | | | D | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 16.25§ | 2.61 | 6.81 | 121.98 | 2.52 | .370 | | 54.5 | 0.75 |
| | | | D | 298 + 24 | 780 + 75 | 48,251 | 53,949 | 16.25§ | 2.61 | 6.81 | 67.69 | 1.40 | .206 | | 75 | 1.01 |
| P-33 | Apr. 18, 1940 | 43.8 | B | 575 + 82 | 780 + 75 | 20,493 | 67,828 | 20.7 | 3.33 | 11.09 | 49.44 | 2.41 | .218 | 7 | 71 | 0.98 |
| P-34 | Apr. 25, 1940 | 43.8 | A | 575 + 82 | 780 + 75 | 20,493 | 67,828 | 22.7 | 3.65 | 13.32 | 59.19 | 2.88 | .216 | 6¶ | 72 | 0.98 |

Banks and Inglee (6, p. 1367) state that it is unlikely that the $C_w$ (Williams-Hazen) of the original line was ever more than 110. After cleaning (before cement lining) a section 12,000 ft. long showed a $C_w$ of 110. Very careful final tests on a lined section 16,000 ft. long showed a $C_w$ of 124. This latter value was based on the original nominal diameter of 48 in. If converted to the diameter of the actual finished inside dimension of the pipe, the coefficient would be 130.

On a basis of identical velocities and nominal coefficients as shown by the curves in Fig. 3, it is difficult to choose between the $C_w$ in 1936 and that in 1940. The obvious conclusion is that the line is as good now as when first relined with cement mortar.

*Recent Physical Condition of the Pipe Lines.*

A description of the No. 2 pipe, as observed by Merriman in 1907, has already been given. In 1932, a portion of this line, about a mile below Macopin Intake, was unwatered. The writer had an opportunity to join the party of engineers who inspected it. On the invert of the pipe, large tubercles, up to perhaps $1\frac{1}{2}$ in. in diameter and frequently an inch in height, existed in large quantities. These tubercles were fairly uniform in density up to a point a few inches above the invert, where they began to thin until at the central horizontal diameter they had materially lessened. Above that point, smaller blisters occurred at intervals, diminishing so that in many places the top of the pipe appeared to have merely a slight roughness. With the pipe running partly full, as was often the case, such a condition seems logical.

The tubercles were all hollow, having a substantial outer wall about $\frac{1}{32}$ in. thick (although in some cases there were thin layers separated by air spaces). This outer layer or wall consisted mainly of pipe coating, with underneath layers of iron and perhaps manganese. Underneath the tubercles, the pipe was eroded to a small depth, probably rarely more than $\frac{1}{16}$ in., or a quarter the thickness of the pipe.

A few hundred feet below the point where it was unwatered the pipe declines very steeply to pass under the Pequannock River. A reliable informant stated that just below the point of steep decline, the location of the larger tubercles was reversed *to the top of the pipe.* Several engineers present at the time discussed this phenomenon and concluded that the scarifying of the surface of the pipe coating

by sand, grit or other materials had paved the way for tuberculation. It was reasoned that when the particles had reached the decline, they had been carried, by inertia, to the top of the pipe, and these had produced conditions similar to those observed on the bottom of the pipe elsewhere. So far as is known, no one has yet offered a

Tuberculation in Newark 60-Inch Pipe From Great Notch—Pipe in Use Over 30 Years

more plausible explanation of the condition. An illustration used by Banks and Inglee (6) showed tuberculation over the entire inside surface and it is possible that large parts of the line were thus affected.

The decision to start a reconditioning program was born of the necessity to increase pipe line capacity to meet peak drafts. The

critical section on both lines is, as stated before, above Pompton Notch. Accordingly, the No. 2 conduit, above that point, was selected for the first large contract for relining. The success of this contract and the early enameling work led to a decision to carry the program further. Under W.P.A. and contracts, the lower 5 mi. (from Bloomfield to Belleville) of each conduit was relined, one with an enameled lining and one with cement mortar. Naturally, this should furnish an unequalled opportunity to compare the two types of coating. Unfortunately, the necessity of maintaining certain conditions during normal operation makes it difficult to obtain suitable pressure differentials to justify observations. Should such an opportunity be afforded, either by permissible or enforced changes in operating routine, many persons concerned will welcome the chance to carry on the tests. The writer saw both types of lining being placed and was impressed with the excellence of both processes. The outstanding feature was the manner in which cement mortar covered rivets. It might be expected that the mortar would not be uniformly applied to uneven surfaces presented by pipe laps and rivets but the final results appeared to exceed all reasonable expectations.

As far as the outside of the pipe is concerned, most of the two lines, when uncovered, have been found to be in a reasonably good state of preservation. At one point, in a golf links, between Great Notch and Belleville, about four miles from the lower end of the line, frequent leaks have occurred. Numerous observations by the Newark Water Department and the writer, made by standard electrolysis methods, have shown the pipe lines to be positive to ground in that vicinity. Uncovering and repairing of these leaks have indicated a condition typical of electrolytic corrosion. It may be said, however, that most of this difficulty was attributed to street car operations. In the last three or four years, most of the trolleys have been replaced by bus transportation, either gas-motored or with two overhead wires, and the change has eliminated the large diffusion of electric current.

Many years ago a trolley line was believed to have caused corrosion in another part of the system (above Great Notch), but the abandonment of the electric line apparently resulted in a cessation of measurable electric currents on that portion. Leaks, which had been numerous, are rarely found in the section now. It is too early to say definitely that all leaks caused by such conditions have ceased.

In the first place, some of the corrosion of the past may have reached a point where, eventually, other natural causes will complete the disintegration of the pipe shell in places. Since the location where most of this occurred has now been relined, probability of leakage has been reduced to a minimum.

It has been shown clearly, by tests, that some of the distribution lines within the city of Newark are affected by current flows on the Delaware, Lackawanna and Western Railroad, which operates on 3,000 volts direct current. Some of this current may be carried back to the transmission conduits. In general, it seems more probable that most of the current is dissipated elsewhere.

Bearing in mind that cement mortar constitutes essentially a comparatively thick lining and bituminous material, a coating—it is reasonable, under present circumstances, to state that, except for occasional leaks that might occur in any such pipe line, the conduits after being relined should be good for at least another 50 years. It is obvious that the effect of external corrosion will be offset materially by relining. This would seem to be particularly true of the cement-mortar lining.

### Rockaway (Jersey City) Conduits

In 1899, the City of Jersey City made a contract with Patrick H. Flynn for a supply of 50 m.g.d. to be obtained from the Rockaway River, with a storage reservoir at Boonton. When the contractor failed, the work was prosecuted by the Jersey City Water Supply Company which was closely affiliated with, and governed by, the same men who had built the Pequannock works. J. Waldo Smith was Chief Engineer.

In this case, the topography lent itself more readily to grade aqueduct construction. The upper half of the line consisted of alternate grade and pressure sections and the lower half was entirely a steel pressure section. Because of the methods pursued, the alignment and the personnel involved, it seems evident that this same type of construction had originally been planned for the Pequannock lines but that, for one or more reasons, the Pequannock design was changed. It might not be amiss to suggest here that a continuous pressure line for Jersey City would have been far preferable from an operating standpoint.

The concrete grade sections were constructed for ultimate capacity but the pressure sections were designed for a lesser capacity initially.

Connections, however, were left for a pressure pipe, 48 in. in diameter, to be laid parallel with the original 72-inch pressure pipe, when required. (When the second pipe was laid later, it was made 72 in. in diameter).

Mills (1, p. 217) gives a few brief but significant comments on this pipe, in referring to tests by Smith in 1903, as follows (for the section experimented on):

Joints tapered; one longitudinal seam, double riveted.
Girth seams, single riveted.

| | |
|---|---|
| Length of 72-inch diameter pipe | 49,061 ft. |
| Length of reducers (72 in. x 48 in.) | 233 ft. |
| Total length tested | 49,294 ft. |

| THICKNESS | RIVET SIZE | LAYING LENGTHS OF PIPE | TOTAL LENGTH |
|---|---|---|---|
| *in.* | *in.* | *ft.* | *ft.* |
| $\frac{5}{16}$ | $\frac{3}{4}$ | 6 | 8,851 |
| $\frac{3}{8}$ | $\frac{7}{8}$ | 7.5 | 7,578 |
| $\frac{7}{16}$ | 1 | 7.5 | 31,315 |
| $\frac{11}{16}$ | $1\frac{1}{8}$ | 5 | 1,317 |
| | | | 49,061 |

Pipe dipped horizontally; Gilsonite coating.
Discharge measured by Venturi meter at downstream end.
Pipe use began May, 1903, but parts were filled many months before, averaging Nov. 1, 1902.

There were ten 48-inch gate valves on the line, but, in two cases (at crossings of the Passaic and Hackensack Rivers), two valves were close together on the upstream side of the crossings. In general, the line valves were about $1\frac{1}{2}$ mi. apart.

The entire pressure section of the lower part of the line was 56,706.8 ft. long, with thicknesses and lengths about as follows:

| THICKNESS OF PIPE | APPROXIMATE LENGTH |
|---|---|
| *in.* | *ft.* |
| $\frac{5}{16}$ | 16,100 |
| $\frac{3}{8}$ | 7,280 |
| $\frac{7}{16}$ | 32,000 |
| $\frac{11}{16}$ | 1,320 |
| | 56,700 |

As taken from the plan of pipe line No. 2, the first line apparently had about 584° of horizontal curvature, or 10.2° per thousand feet. This was made up of 28 curves, mostly of 200-foot radius. Mills (1, p. 217) gives the number of curves as 21, and probably this was correct for the section tested and recorded in his book. He also mentions 94 vertical changes in direction, 28 air valves, 15 blow-offs, and 53 manholes in the entire lower section of the line. A map on file shows about 100 vertical changes in direction, 28 air valves, 15 blow-offs and 62 manholes.

The invert elevation at the upper end of the section was 229.4, falling to 117.7 at the lower end. The actual drop was 111.7 ft., or 1.98 ft. per thousand feet. The total vertical drop was about 583 ft. and total rise 471 ft., giving a gross change in elevation of 1,054 ft., or 18.6 ft. per thousand feet of length.

In some cases, the degrees of cut (or vertical angle deviation) were given on the drawings of the line. By summing up those indicated and estimating for the balance, it appears that the vertical curvature was around 500°. While this was somewhat less than the horizontal curvature, in general it was accomplished with much smaller radii and, therefore, was likely to have a considerably greater effect on the carrying capacity of the pipe.

The second pipe was started in 1919, but was made 72 in. in diameter instead of 48 in. as originally planned. A report of the National Board of Fire Underwriters, dated Nov. 12, 1919, says:

"Work is well under way installing duplicate 72-inch steel mains under the Passaic and Hackensack rivers. A contract has been let to duplicate the steel sections. . . from the waste weir at the mountain to the Passaic River."

The writer recalls seeing large shipments of this pipe being delivered at the job and then remaining there for some time. This checks with the statement in *Aquafax* (1925) which says that the extension was completed in 1921.

The second pipe paralleled the first precisely, except for locations where offsets (usually in tandem) had to be made in the second line. The new line was of lock-bar, in 30-foot sections—two lock-bars per section with single button-head riveted girth joints. From the specifications for the second pipe, dated 1919, the following is abstracted:

> After test, pipe cleaned, heated to 350°F., dipped in hot bath, $\frac{1}{32}$-inch minimum coating of Hermastic; coat to be free

from blisters, waterproof, not affected by salts, acids, heat or cold; recoated if defective and protected during riveting by a strip of canvas, men wearing rubber-soled shoes.

Thicknesses and lengths of this second line were as follows:

| THICKNESS | LENGTH |
|---|---|
| *in.* | *ft.* |
| $\frac{3}{8}$ | 21,262 |
| $\frac{7}{16}$ | 31,864 |
| $\frac{11}{16}$ | 250 |
| $\frac{9}{16}$ | 1,405 |
| $\frac{3}{4}$ | 1,794 |
| Total | 56,575 |

It should be noted that this length includes reducers and gate valves.

There were three gate valves in normal locations along the line and seven others at river crossings. All valves were 48-inch, located as follows:

| *Station* | *Location (if special)* |
|---|---|
| 707 + 51 | |
| 770 + 39 | |
| 827 + 36 | West bank of Passaic River |
| 836 + 72 | East bank of Passaic River |
| 838 + 19 | East bank of Passaic River |
| 1018 + 78 | West bank of Hackensack River |
| 1018 + 89 | West bank of Hackensack River |
| 1032 + 39 | East bank of Hackensack River |
| 1034 + 18 | East bank of Hackensack River |
| 1112 + 31 | |

This gave the same number of valves as line No. 1 but at different locations. In only one of these cases, on the west bank of the Hackensack River where the two valves were quite close, was the 48-inch diameter continued through both valves.

While the vertical rise and fall in this line were almost identical with those of the No. 1 line, the horizontal curvature, because of numerous offsets, totaled 982°, or 17.3° per thousand feet as compared with 10.2° for the No. 1 line.

*Tests on Rockaway Pipe Lines.*

Mills (1) recorded observations by J. Waldo Smith on the first of these lines; and Scobey (3) not only included these, but supplied a number of additional results. It should be noted that Scobey's year

and age notations do not check, but from information available, it seems that the ages given are correct. This indicates a series of tests that began in 1903 and ended in 1915.

In 1919, and again in 1925, further observations were made. The latter set was conducted by Paul Lanham of Washington, D. C.;

72-Inch Conduit of ¼-Inch Riveted Steel Installed in Jersey City in 1902; sandblasted for relining about 32 years later

and he may also have been identified with those of 1919, as well as others. City officials and the National Board of Fire Underwriters played a large part in these undertakings and the reports of the latter body furnish the only available record of the observations. The 1925 results include the only known printed records on the No. 2 line.

In 1935, Harold M. Ohland, Hydraulic Engineer of the Jersey City Water Bureau, inspected both lines in the pressure sections, a few thousand feet upstream from the test sections. He has told the writer that the No. 1 conduit was fairly profuse in tubercles but not, apparently, as much so as the decrease in capacity would indicate. The No. 2 conduit was found in much better condition and its capacity, it is believed, has not dropped greatly from the original value. In the vicinity of these inspections, a stretch of the No. 1 line, several thousand feet in length, was first cleaned and then relined with a bitumastic lining. Observations were made on a short section of the pipe in 1935 and 1936, before and after lining, and the writer is indebted to Mr. Ohland not only for a summary of these, but also for an opportunity to study a circular piece of the

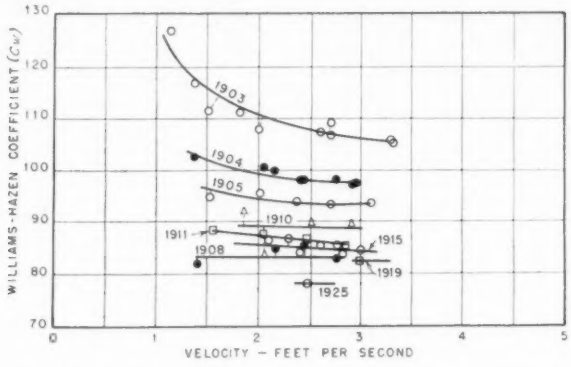



FIG. 8. Variation of Coefficient ($C_w$); 72-Inch Rockaway Conduit No. 1

pipe removed when a wet connection was made early in September, 1940. The piece showed the coating to be virtually as good as new in spite of the hand methods used in its application to the pipe surface. Of course, it is impossible to secure a glass type surface by hand methods, but one cannot help being impressed by the evident improvement in coating materials.

*Results of Tests.*

In construction and deterioration, the No. 1 Rockaway conduit resembles the Pequannock conduits very closely. Figure 7 shows the relation of $C_w$ to age for this line and indicates the close agreement with the Pequannock results, even though the coatings were different. Figure 8 gives the relation of the coefficient to velocity; and here again the similarity to Pequannock results is apparent.

Since all these tests are on the same general section, and at the lower end of the aqueduct, there is no means to observe the effect of distance from the reservoir. In Fig. 7 no curve can be drawn for the second pipe line, because only one set of tests is available, i.e. that of 1925. Further tests on the line, it is expected, will be made in the near future. The two lines are capable of carrying the full yield of the watershed in a dry season and, therefore, there has been no immediate necessity for determining present pipe line coefficients.

Table 6 shows the results of tests subsequent to those published by Scobey.

### Wanaque Aqueduct

Need for additional water for Newark and other municipalities resulted in the joint Wanaque project. Under the agency of the North Jersey District Water Supply Commission, work was started for the City of Newark in December, 1920. Eventually Paterson, Kearny, Passaic, Clifton, Montclair, Bloomfield, and Glen Ridge also became participants.

The system was nominally designed for 100 m.g.d. and the aqueduct was planned to operate at grade, with construction quite similar to that of the Jersey City pipe line. Eventually, the design was changed to provide a pressure aqueduct; and the two tunnels, already constructed, were reinforced to permit operation under pressure.

A more complete understanding of the various portions of this line, will be given by Fig. 9 which shows the profile and the sizes of the pipe used. Figure 10 is a general plan of all the large conduits, those already described as well as some others that will be mentioned later. The figure will serve to show the relationship of these lines to each other.

Unlike the Pequannock and Jersey City lines, which operate at gravity, the Wanaque aqueduct has a pumping station at the upper end and others at two of the four points of the take-off. It has never been necessary to operate the head-works pumping station, although it was very nearly so early in 1940, when the reservoir level was so low that pressures at the lower end were close to the minimum permissible.

As may be seen from Figs. 9 and 10, the upstream two-thirds of the aqueduct consists of twin 74-inch diameter lines. These were made of 30-foot lock-bar sections, each section having two lock-bars

TABLE 6

PART 1. *Tests on 72-Inch Rockaway Conduit No. 1*

Riveted steel: 7½-foot lengths; Nominal $D$ = 6.0 ft.; $D^{1.25}$ = 9.390; $D^{0.625}$ = 3.064

| TEST NO. | DATE | AGE OF PIPE | $L$ LENGTH OF TEST SECTION | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = M'$ | $C_w$ WILLIAMS-HAZEN | $F_m$ MILLS | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Total | Per thousand feet $H$ | | | | |
| | | *years* | *feet* | *m.g.d.* | *feet per second* | | *feet* | *feet* | | | | |
| R-1 | 1915 | 13 | | | | | | | | 85 | | No details given |
| R-2 | 1919 | 17 | 56,663.5 | 54 | 2.96 | 8.76 | 44.47 | 0.786 | 0.0897 | 82 | 1.09 | |
| R-3 | 1925 | 23 | 56,663.5 | 45.5 | 2.49 | 6.20 | 35.63 | 0.630 | 0.1013 | 78 | 1.04 | |
| R-4 | 1935 | 33 | 625 | | | | | | | 80 | | Before relining |
| R-5 | 1936 | 34 | 625 | | | | | | | 102 | | After relining |

PART 2. *Tests on 72-Inch Rockaway Conduit No. 2*

Lock-Bar: 30-foot lengths; Nominal $D$ = 6.0 ft.

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| R-6 | 1925 | 4 | 56,706.8 | 58 | 3.17 | 10.05 | 33.32 | 0.589 | 0.0586 | 103.5 | 1.35 | 24-hour run |
| R-7 | 1925 | 4 | 56,706.8 | 72 | 3.94 | 15.52 | 56.39 | 0.997 | .0642 | 96.7 | 1.29 | 4-hour run |
| R-8 | 1925 | 4 | 56,706.8 | 78 | 4.27 | 18.23 | 56.39 | 0.997 | .0547 | 104.5 | 1.40 | Peak rate |

NOTE: Test No. R-1 may be same as Scobey's 64f. All tests except R-4 and R-5 located between Montclair and Jersey City, about Sta. 639 + 00 to 1207 + 00. Tests R-4 and R-5 located near Sta. 300 + 00.

to join its two sheets together. Button-head girth rivets were in a single row. The lower portion of the aqueduct (below the Great Notch tunnel) was constructed of a similar single 74-inch diameter line.

Because of the location of certain gaging stations and because of variations in flow and diameter, tests made on the aqueduct have been largely confined to an upper stretch between 59,000 and 65,000 ft. in length and a lower section 23,569 ft. long. Curvature

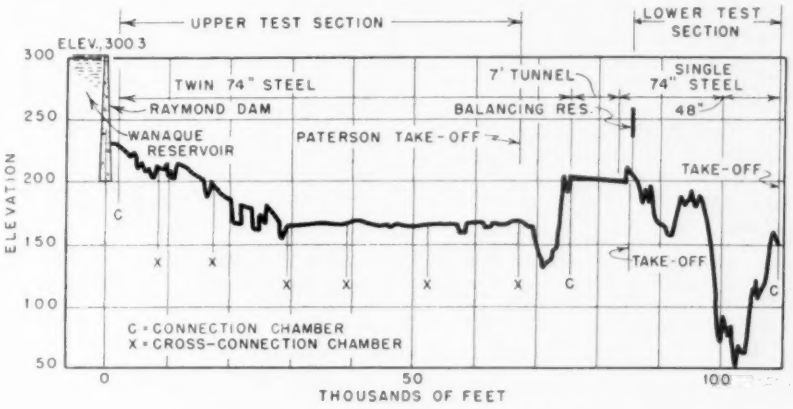



FIG. 9. Profile of Wanaque Aqueduct

on the upper 64,725 ft. of twin line (the section most readily and frequently tested) is divided as follows:

| NUMBER OF CURVES | RADIUS IN FEET | TOTAL CURVATURE |
|---|---|---|
| 1 | 100 | 83°39′ |
| 8 | 200 | 364°19′ |
| 1 | 350 | 49°07′ |
| 1 | 450 | 32°00′ |
| 25 | 500 | 645°05′ |
| 15 | above 500 | 398°01′ |
| 3 | not given | 10°43′ |
| 54 | | 1,582°54′ |

Thus the horizontal curvature is 24.6° per thousand feet of pipe.

For the lower 23,569 ft., the horizontal curvature is 290°09′, or 12.3° per thousand feet of pipe, being about equally divided between curves with radii of 25 ft., 500 ft. and 2,300 ft.

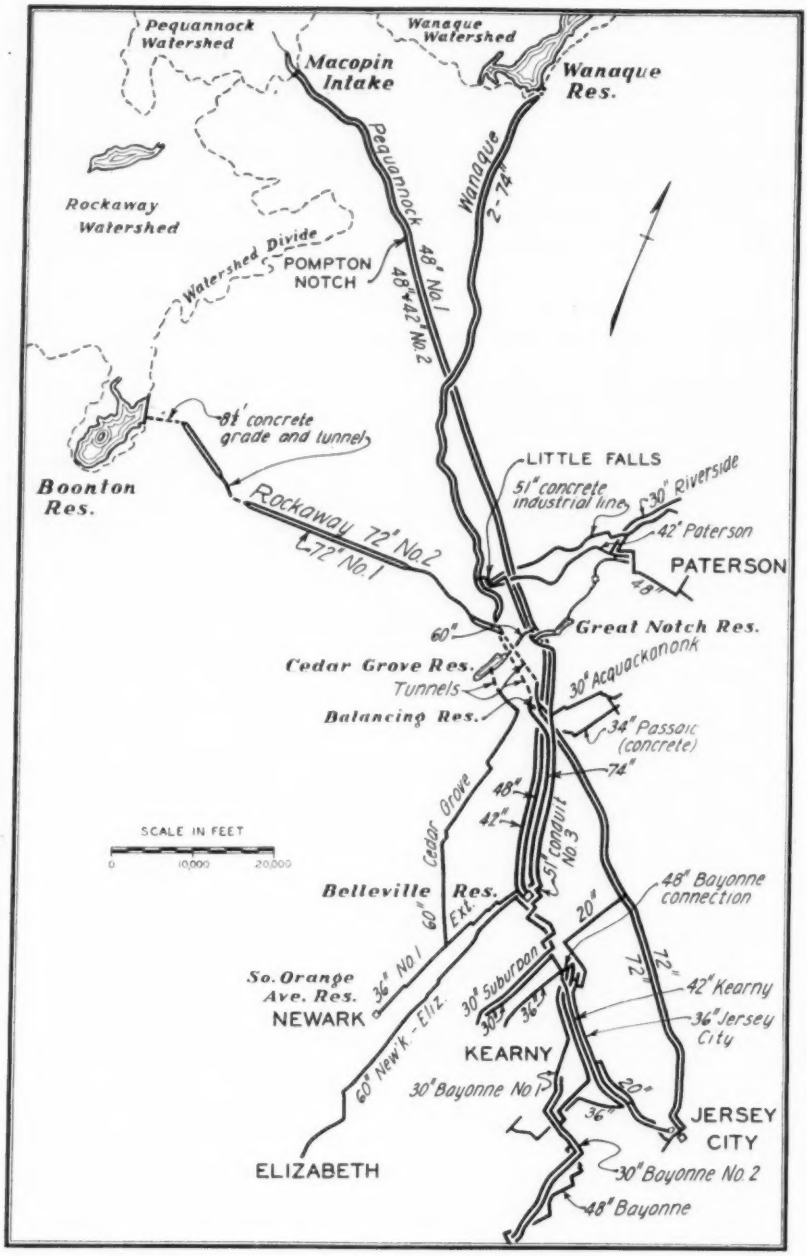



FIG. 10. Transmission Mains; Surface Supplies—Passaic River Basin

Over the entire aqueduct the curvature is 2,282°17′ for a length of 108,902 ft. or 21° per thousand feet length.

Vertical changes are as follows:

| | UPPER SECTION OF AQUEDUCT | LOWER SECTION OF AQUEDUCT |
|---|---|---|
| Length | 64,725 ft. | 23,569 ft. |
| Drop of invert | 60 ft. | 55.4 ft. |
| Total drop | 233.9 ft. | 292.6 ft. |
| Total rise | 173.9 ft. | 237.2 ft. |
| Total change in elevation | 407.8 ft. | 529.8 ft. |
| Total change per thousand feet | 6.3 ft. | 22.4 ft. |
| Changes in vertical direction | 102* | 82† |
| Changes in vertical direction per thousand feet | 1.59 | 3.48 |

* Exclusive of cross-connection chambers.

† Exclusive of reduced section.

In the design of this aqueduct, much thought was given to the probability of decrease in carrying capacity with age. It was first recommended that valves be spaced at 1-mile intervals, primarily for the purpose of isolating a section so that it could be cleaned and refinished periodically. Finally, the spacing was set at about 2 mi. in the upper portion of the line. No line valves were placed in the lower section between Great Notch Tunnel and Belleville, a distance of nearly 5 mi.

Partly because of this problem of cleaning the aqueduct, five gates were installed at each of the points of cross-connection, except at the ends of the twin lines, where four gates were used. Figure 11 shows a typical cross-connection chamber. The writer spent some time in studying the question of size, number, and type of valves. In the light of information then available, the use of 48-inch valves (as adopted) seemed to be consistent with economy, ease of operation, and loss of head. Subsequent tests on losses in these chambers led to the belief that the ratio of the diameter of the valve to the pipe diameter should be as near unity as possible wherever loss of head is an important factor.

In early computations of losses in this aqueduct, both in the early grade and latter pressure designs, values of Kutter's $n$ of 0.017 and even 0.019 were considered reasonable for steel pipe. These were intended to represent values existing after a considerable period of operation. Later a range of 0.013 to 0.015 was used, corresponding

to a $C_w$ (Williams-Hazen) of 119 to 107. Results of tests (given later) show to what extent these values were justified.

All of the steel pipe is $\frac{7}{16}$ in. thick with exception of portions between stations 1007 + 09 to 1011 + 07 and 1020 + 18 to 1040 + 09, a total distance of 2,389 ft., where the thickness is $\frac{1}{2}$ in. In this portion (located within the lower section tested), a low elevation of invert of 54.0, representing a maximum static head of nearly 250 ft., is reached.

In this same lower section there occurs the only reduction in size not located at a cross-connection chamber. In crossing a street,

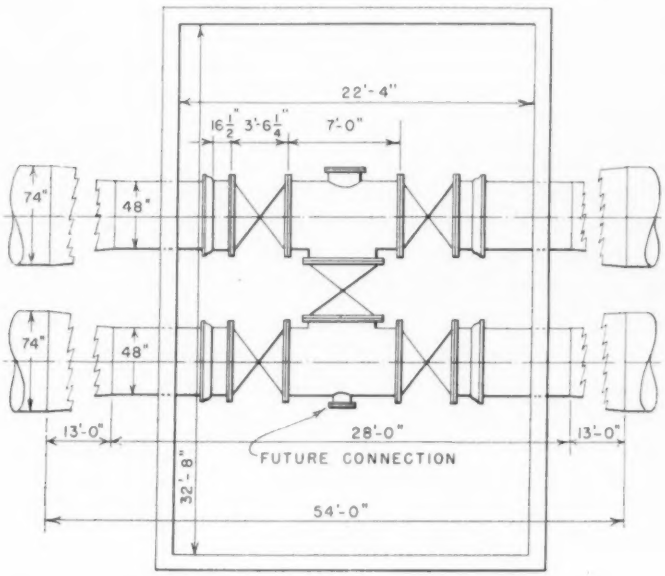



FIG. 11. Cross-Connection Chamber; Wanaque Aqueduct

several utilities were encountered and it was decided to introduce a section 10 ft. long and 48 in. in diameter, with a 74 in. by 48 in. reducer, 15 ft. long, at each end. This permitted passing between existing utilities instead of going beneath them in wet ground. The plate is $\frac{7}{16}$ in. thick at this location.

On the upper section tested there are 33 air valve chambers, of which 3 have two 6-inch valves, 22 have one 6-inch valve, and 8 have one 4-inch valve. In this same section there are 30 drainage manholes, of which 13 have 12-inch blow-offs and 17 have 8-inch blow-offs; and in the same distance there are 41 manholes.

On the lower section tested there are 17 valve chambers, of which 1 has eight 6-inch valves, 6 have three 6-inch valves, 3 have two 6-inch valves, 3 have one 6-inch valve and 4 have one 4-inch valve. All but one of the chambers have one or more pressure relief valves. In this same section, there are 12 blow-offs, of which 4 have 12-inch and 8 have 8-inch blow-offs. Manholes in the section total 23.

Construction of the line was started January 24, 1929 and was completed June 25, 1930 by the T. A. Gillespie Co. which had also laid the other lines previously described. Because of the need for fulfilling contractual obligations for water, operations began officially on March 20, 1930. For purposes of estimating the period of use,

Wanaque Aqueduct (1929) Curve Showing Buttresses

it is sufficient to take January 1, 1930 as about the average time of filling the aqueduct.

Abstracted requirements from the specifications are as follows:

Pipe cleaned, heated to 350°F., dipped in hot bath (450°F.) of Hermastic or Ovarco (latter was used). Coating free from blisters, not affected by heat or cold, $\frac{1}{32}$ in. minimum thickness. Pipe wrapped with Pabco, when sufficiently cooled, under tension of 50 lb. per 18-inch width. Special care to protect coatings, men to wear rubber-soled shoes. Leakage not to exceed 8,500 gal. per mi. of pipe per 24 hr. After field tests, pipe dewatered and abrasions recoated.

TABLE 7.—*Tests on 74-Inch Wanaque Pipes*

Lock-Bar Pipe: 30-foot lengths; Nominal $D$ = 6.167 ft.; $D^{1.25}$ = 9.718; $D^{0.625}$ = 3.117

| TEST NO. | DATE | AGE OF PIPE | OBSERVATION RATING | PERIOD OF TEST TIME TO TIME | | LOCATION OF TEST SECTION STATION TO STATION | | $L$ LENGTH OF TEST SECTION | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD Total | LOSS OF HEAD Per thousand feet $H$ | $\frac{H}{V^2} = M'$ | TEMPERATURE OF WATER | $C_W$ WILLIAMS-HAZEN | $F_m$ MILLS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *years* | | | | | | *feet* | *m.g.d.* | *ft./sec.* | | *feet* | *feet* | | °C. | | |
| W-1* | Aug. 25 and 29, 1930 | 0.6 | D | 10.00 | 10.30 a.m. | 19 + 06 | 669 + 84 | 65,078 | 24.5 | 1.27 | 1.61 | 5.7 | 0.0875 | 0.0545 | 13 | 114 | 1.36 |
| | | | | 2.00 | 2.30 p.m. | | | | | | | | | | | | |
| W-2† | Oct. 27, 1930 | 0.8 | D | 10.00 | 10.30 a.m. | 19 + 06 | 669 + 84 | 65,078 | 44.0 | 2.28 | 5.20 | 16.21 | 0.250 | .0480 | 12 | 116 | 1.48 |
| W-3† | Oct. 27, 1930 | 0.8 | D | 10.30 | 11.00 a.m. | 19 + 06 | 669 + 84 | 65,078 | 49.0 | 2.54 | 6.45 | 19.67 | 0.303 | .0469 | 12 | 116 | 1.48 |
| W-4 | Aug. 18, 1930 | 0.6 | C | 10.30 | 11.00 a.m. | 840 + 99 | 1089 + 36 | 24,837 | 44.4 | 2.28 | 5.20 | 4.94 | 0.198 | .0380 | 12 | 132 | 1.65 |
| W-5 | Oct. 27, 1930 | 0.8 | C | 10.00 | 10.30 a.m. | 840 + 99 | 1089 + 36 | 24,837 | 39.8 | 2.06 | 4.24 | 3.78 | 0.152 | .0359 | 12 | 137 | 1.70 |
| W-6‡ | Oct. 27, 1930 | 0.8 | D | 10.30 | 11.00 a.m. | 840 + 99 | 1089 + 36 | 24,837 | 39.6 | 2.05 | 4.20 | 6.32 | 0.254 | .0604 | 12 | 106 | 1.31 |
| W-7† | Mar. 11, 1931 | 1.2 | A | 10.00 | 10.30 a.m. | 76 + 50 | 669 + 24 | 59,274 | 70.5 | 3.65 | 13.32 | 31.18 | 0.526 | .0395 | 3 | 124.4 | 1.62 |
| W-8† | Apr. 11, 1931 | 1.3 | A | 10.00 | 10.30 a.m. | 76 + 50 | 669 + 24 | 59,274 | 72.5 | 3.76 | 14.14 | 32.69 | 0.5515 | .0390 | 5 | 124.7 | 1.63 |
| W-9† | Apr. 13, 1931 | 1.3 | A | 10.00 | 10.30 a.m. | 76 + 50 | 669 + 24 | 59,274 | 72.5 | 3.76 | 14.14 | 32.56 | 0.5493 | .0388 | 6 | 125.0 | 1.63 |
| W-10 | Mar. 11, 1931 | 1.2 | B | 9.00 | 12.00 a.m. | 853 + 65 | 1089 + 34 | 23,569 | 39.0 | 2.02 | 4.08 | 3.96 | 0.1680 | .0412 | 3 | 127.4 | 1.59 |
| W-11§ | Aug. 8, 1939 | 9.6 | B | — | 10.30 a.m. | 21 + 99 | 669 + 24 | 64,725 | 51.25 | 2.66 | 7.08 | 19.97 | 0.3085 | .0436 | 13 | 120.6 | 1.54 |
| | Aug. 8, 1939 | 9.6 | C | — | 11.00 a.m. | 21 + 99 | 669 + 24 | 64,725 | 50.75 | 2.63 | 6.92 | 19.97 | 0.3085 | .0446 | 13 | 119.4 | 1.52 |
| | Aug. 8, 1939 | 9.6 | A | — | 11.30 a.m. | 21 + 99 | 669 + 24 | 64,725 | 49.50 | 2.56 | 6.55 | 18.36 | 0.2837 | .0434 | 13 | 121.9 | 1.54 |
| | Aug. 8, 1939 | 9.6 | A | — | 12.00 n. | 21 + 99 | 669 + 24 | 64,725 | 48.25 | 2.50 | 6.25 | 17.43 | 0.2693 | .0430 | 13 | 122.2 | 1.55 |
| | Aug. 8, 1939 | 9.6 | A | 10.00 | 10.30 a.m. | 21 + 99 | 669 + 24 | 64,725 | 51.25 | 2.66 | 7.08 | 19.58 | 0.3025 | .0428 | 13 | 121.9 | 1.56 |
| | Aug. 8, 1939 | 9.6 | C | 10.30 | 11.00 a.m. | 21 + 99 | 669 + 24 | 64,725 | 51.33 | 2.66 | 7.08 | 20.34 | 0.3143 | .0444 | 13 | 119.6 | 1.55 |
| | Aug. 8, 1939 | 9.6 | B | 11.00 | 11.30 a.m. | 21 + 99 | 669 + 24 | 64,725 | 49.78 | 2.58 | 6.66 | 18.54 | 0.2864 | .0430 | 13 | 122.0 | 1.55 |
| | Aug. 8, 1939 | 9.6 | A | 11.30 | 12.00 a.m. | 21 + 99 | 669 + 24 | 64,725 | 49.34 | 2.56 | 6.55 | 18.52 | 0.2861 | .0437 | 13 | 121.0 | 1.54 |
| | Aug. 8, 1939 | 9.6 | B | 10.00 | 12.15 p.m. | 21 + 99 | 669 + 24 | 64,725 | 50.12 | 2.60 | 6.76 | 18.98 | 0.2932 | .0434 | 13 | 121.2 | 1.54 |
| W-12 | Aug. 8, 1939 | 9.6 | B | 10.10 | 12.10 a.m. | 853 + 65 | 1089 + 34 | 23,569 | 60.36 | 3.13 | 9.80 | 9.36 | 0.3971 | .0405 | 13 | 124.0 | 1.60 |

* Tests of two dates combined.
† Only one pipe in use.
‡ Part of flow to balancing reservoir.
§ Both pipes in use.

*Tests on Wanaque Aqueduct.*

Unfortunately the need for placing this line in service and the rush to complete other parts of the system were so great that no tests were made during the early days of operation. The first recorded tests were made in August and October, 1930. These shed some light on the subject but they cannot be regarded as having great accuracy. A summary of results is given in the first portion of Table 7.

Large variations in the coefficient in the lower part of the line are probably due, in part, to approximate elevations and readings.

Wanaque Aqueduct (1929) Cross-Connection Chamber

Variations in flow in this portion of the line, particularly in estimates of the volume discharged into the balancing reservoir, also served to influence results. Only one value of $C_w$, that of August 18, is fairly consistent with later results.

In March, April, and May, 1931, an excellent series of tests was made. Table 7 shows the results of these observations. In each of these three 1931 cases, all the flow in the upper (twin) portion of the aqueduct was routed through one pipe. Pressures were obtained by Ashcroft gages, $8\frac{1}{2}$ in. in diameter, carefully calibrated before and after the run on a Crosby dead weight testing apparatus.

During the March and April tests, an effort was made to check the loss through the cross-connection chambers. Because of the

* Tests of two dates combined.
† Only one pipe in use.
‡ Part of flow to balancing reservoir.
§ Both pipes in use.

inability of the gages employed to record accurately losses of this order, an additional set of observations was made on May 4, 1931. An inverted U-tube was employed and connections were made through hose lines to the pipe line on each side of the cross-connec-

TABLE 8

*Loss of Head Through Cross-Connection Chambers; Wanaque Aqueduct*

Tests on May 4, 1931 by Joseph A. Ward; flow of 72 m.g.d. maintained through east line, 9 A.M. to 12 NOON; temperature of water 9.0° C.

| (1) CENTER LINE OF CHAMBER AT STA. 393 + 16 | | | (2) CENTER LINE OF CHAMBER AT STA. 524 + 34 | | | (3) CENTER LINE OF CHAMBER AT STA. 670 + 24 | | |
|---|---|---|---|---|---|---|---|---|
| Time, A.M. | Sta. 391 + 11 | Sta. 394 + 11 | Time, A.M. | Sta. 523 + 44 | Sta. 525 + 74 | Time, A.M. | Sta. 669 + 24 | Sta. 671 + 74 |
| | Height of water column in feet | | | Height of water column in feet | | | Height of water column in feet | |
| 9:05 | 1.92 | 1.25 | 9:55 | 1.88 | 1.11 | 10:30 | 1.64 | 0.89 |
| 9:06 | 1.98 | 1.23 | 9:56 | 1.86 | 1.10 | 10:31 | 1.64 | .89 |
| 9:07 | 1.98 | 1.21 | 9:57 | 1.86 | 1.10 | 10:32 | 1.63 | .88 |
| 9:08 | 2.00 | 1.17 | 9:58 | 1.87 | 1.10 | 10:33 | 1.63 | .89 |
| 9:09 | 2.03 | 1.17 | 9:59 | 1.86 | 1.10 | 10:34 | 1.64 | .89 |
| Average | 1.98 | 1.21 | | 1.86 | 1.10 | | 1.64 | 0.89 |
| Loss in head | 0.77 | | | 0.76 | | | 0.76 | |
| Length = 300 ft. | | | Length = 230 ft. | | | Length = 250 ft. | | |

| | | |
|---|---|---|
| Average distance between gaging manholes | 260 | ft. |
| Average loss of head | 0.763 | " |
| Total length of specials at each chamber | 54 | " |
| Length of straight pipe between gages | 206 | " |
| Loss (at this velocity) for straight pipe | 0.54 | " per thousand |
| Loss in straight pipe (per chamber) = 0.206 × 0.54 | 0.111 | " |
| Net loss in specials = 0.763 − 0.111 | 0.652 | " |
| Velocity head (for velocity of 3.73 ft. per sec.) | 0.216 | " |
| Loss through specials = 3.02 velocity heads or approximately | $3\frac{V^2}{2g}$ | |

tion chambers. Results of this special set of readings are given in Table 8.

In order to allow correctly for the length of straight pipe included, the loss per thousand feet, as indicated by the tests of March and April, 1931, was calculated and found to be about 0.54 ft. per thou-

sand for the velocity then encountered. For an average length of straight pipe of 204 ft., the loss was 0.204 x 0.54, or 0.11 ft. Deducting this from the gross loss of 0.76 ft., as shown in Table 8, gives a net loss of 0.65 ft. through the cross-connection chamber and reducers. This loss represents about three times the velocity head, being much greater than was estimated originally on the basis of information available at the time the line was designed. The obvious conclusion is that losses of this nature must be given greater consideration.

Based on the results of Tables 7 and 8, it was concluded that the pipe line had exceeded the original expectations. Nevertheless, most engineers believed that a steady deterioration would gradually reduce the carrying capacity. Engineers who had part in the operation of the line were unable to obtain consent to make arrangements for further periodical tests until the impetus of the interconnection study indicated that such observations were essential to establish the carrying capacity of all lines in the vicinity. An initial set of observations was made on April 6, 1939, and a second set on August 8, 1939.

The upper sections observed had an upstream point about 1 mi. above that of 1931, but the lower point was identical, as were both points on the lower section. Various difficulties encountered on April 6, 1939, make the usefulness of the results doubtful. On the second occasion (August 8) all possible precautions were taken to obtain reliable records. Venturi meter readings, both indicated and total, were taken at 5-minute intervals at the upstream master meter and at each of the four take-offs. (It should be noted that there were no take-offs between the points of observation.) These were later all reconciled and adjusted to balance. Pressure gages were 3-inch and $3\frac{1}{2}$-inch Bourdon type, tested in the office before the run and *in situ* after the run. The latter was accomplished by having trained men travel along the line at the conclusion of the run, testing each gage with a dead weight tester immediately after removing it from its connected position. Table 7 shows the recorded results and normal values of $C_w$. Values of $C_w$ corrected for reduced sections in the line are given in Table 9, but it is to be remembered that caution must be used in comparing such results with other tests.

In order to compare these findings with those of the two previously described pipe lines, the relation of $C_w$ to age of pipe for both sections of the line tested has been added to Fig. 7, together with records of other lines.

As a result of the tests on the line it may definitely be concluded that the carrying capacity has exceeded expectations. Furthermore, the decrease has been much smaller than on many other lines

TABLE 9

*Effect of Constrictions on Coefficient; Wanaque Aqueduct*

| ITEM | UNIT | TESTS OF MAR. 11, 1931 | | TESTS OF AUG. 8, 1940 | |
|---|---|---|---|---|---|
| | | Upper section | Lower section | Upper section | Lower section |
| Velocity | Ft. per sec. | 3.65 | 2.02 | 2.60 | 3.13 |
| Velocity head $\left(\frac{V^2}{2g}\right)$ | Feet | 0.207 | .0633 | 0.105 | 0.152 |
| No. of velocity heads lost in constrictions | | 15 | 1.5* | 15 | 1.5* |
| Total constriction loss | Feet | 3.105 | 0.095 | 1.575 | 0.228 |
| Gross loss of head | " | 31.18 | 3.96 | 18.98 | 9.36 |
| Net " " " straight pipe | " | 28.08 | 3.86 | 17.40 | 9.13 |
| Full length of pipe and constrictions | " | 59,274 | 23,569 | 64,725 | 23,569 |
| Length of constricted sections | " | 270 | 40 | 270 | 40 |
| Net length of straight pipe | " | 59,004 | 23,529 | 64,455 | 23,529 |
| Loss of head per thousand feet—full length | " | 0.5260 | 0.1680 | 0.2932 | 0.3971 |
| Loss of head per thousand feet—straight pipe | " | 0.4759 | 0.1640 | 0.2700 | 0.3880 |
| Williams-Hazen coefficient—full length | | 124.4 | 127.4 | 121.2 | 124.0 |
| Williams-Hazen coefficient—straight pipe | | 131 | 129 | 127 | 125 |

* Estimated.

in the vicinity. It speaks well for later methods employed in construction and lining. Relining operations, as originally planned, may be postponed, even though not wholly disregarded.

### Other Steel Conduits in Northeastern New Jersey

A number of other steel conduits have been installed in this area, mostly by the East Jersey Pipe Co. Until 1905, most, if not all, of these were similar to the second Pequannock line; while from 1905 on, the bulk of the pipe was of lock-bar construction in 30-foot lengths with a single row of button-head rivets at girth joints. For the sake of completing this record, there is given in Table 10 a summary of all the major steel lines transporting water from the large surface sources controlled originally by the East Jersey Water Co. or its affiliates.

Some tests have been made on the lines not already discussed. The earlier ones have been recorded by Herschel (5); and both Mills (1) and Scobey (3) have republished them. For various reasons, most of these other lines have not been investigated as thoroughly as the Pequannock, Rockaway and Wanaque lines. Many of the early ones had an initial $C_w$ of about 110 which has since dropped to about 75. Most of those installed since 1920, however, are believed to compare favorably with the Wanaque line, *i.e.* having an initial $C_w$ between 120 and 130 and not changing rapidly.

### Passaic Valley Water Commission Concrete Industrial Line

This commission represents the three cities of Paterson, Passaic, and Clifton which became partners in the joint Wanaque project. In 1930, having taken over the Little Falls plant originally built on the Passaic River by the East Jersey Water Co. and later owned by the Passaic Consolidated Water Co., the Commission began to revise the entire system to make the Wanaque water primarily a potable supply, and the Passaic River water an industrial source. This required the construction of a new pipe line from Little Falls to Paterson, a distance of nearly 4 mi. At Paterson it joined the 48-inch steel Dyer's main, shown in Table 10.

After much deliberation the line was installed as a 51-inch lock-joint concrete pipe. The entire length of concrete pipe is 20,542 ft. It conducts filtered Passaic River water to the large dyeing industries in Paterson and vicinity. A very drastic decrease in some of the dyeing processes, that formerly required large quantities of water, has greatly reduced the value of the pipe from an investment standpoint.

TABLE 10

*Principal Steel Pipe Lines in Area Served From Supplies Known as: 1—Pequannock; 2—Rockaway; 3—Wanaque; 4—Passaic River, Filtered; 5—Wanaque, Filtered*

| YEAR INSTALLED | FROM | TO | KNOWN AS | CONSTRUCTED FOR | NOW OWNED BY | DIAMETER | LENGTH | PRESENT SOURCE OF WATER CARRIED |
|---|---|---|---|---|---|---|---|---|
| | | | | | | *inches* | *feet* | |
| 1892 | Macopin | Belleville | Pequannock No. 1 | East Jersey W. Co. | Newark | 48 | 111,882 | 1 |
| 1892 | Belleville | Newark | " " " | " " " " | " | 36 | 24,545 | 1 |
| 1895 | " | Kearny | Kearny Exten. | " " " " | Passaic Valley W. Comm'n | 42 | 8,750 | 5 |
| 1896 | Macopin | Belleville | Pequannock No. 2 | " " " " | Newark | 48 and 42 | 111,704 | 1 |
| 1896 | Kearny | Bayonne | Bayonne No. 1 | N. Y. and N. J. W. Co. | Bayonne | 30 | 36,000 | 3 |
| 1898 | Little Falls | Paterson | Paterson Exten. | East Jersey W. Co. | Passaic Valley W. Comm'n | 42 | 18,600 | 5 |
| 1901 | " " | Belleville | Conduit No. 3 | " " " " | Passaic Valley W. Comm'n | 51 | 43,400 | 5 |
| 1902 | Boonton | Jersey City | Jersey City No. 1 | Jersey City W. Supply Co. | Jersey City | 72 | 93,000 | 2 |
| 1903 | Cedar Grove | Newark | Cedar Grove line | Newark | Newark | 60 | 39,300 | 1 |
| 1903 | Kearny | Harrison | Suburban " | N. J. Suburban W. Co. | Passaic Valley W. Comm'n | 30 | 12,000 | 5 |
| 1905 | Great Falls | Paterson | Dyer's " | Passaic W. Co. | Passaic Valley W. Comm'n | 48 and 42 | 11,000 | 4 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 1910 | Clifton | Passaic | Acquackanonk " | Acquackanonk W. Co. | Passaic Valley W. Comm'n | 30 | 16,400 | 5 |
| 1912* | Kearny | Bayonne | Bayonne No. 2 | N. Y. and N. J. W. Co. | Bayonne | 30 | 36,000 | 3 |
| 1920 | " | " | " 48 | Bayonne | " | 48 | 44,000 | 3 |
| 1921 | Boonton | Jersey City | Jersey City No. 2 | Jersey City | Jersey City | 72 | 88,000 | 2 |
| 1922 | Great Falls | Paterson | Riverside Line | Passaic W. Co. | Passaic Valley W. Comm'n | 30 | 12,000 | 5 |
| 1923 | In Kearny | | | Kearny | Kearny | 30 | 5,600 | 3 |
| 1924 | " " | | | " | " | 30 | 5,150 | 3 |
| 1926 | " " | | | " | " | 36 | 4,300 | 3 |
| 1927 | " " | | | " | " | 30 to 48 | 23,677 | 3 |
| 1928 | Belleville | Kearny | Kearny Connection | " | " | 48 | 8,650 | 3 |
| 1929 | In Kearny | | Bayonne Connection | " | N. J. D. W. S. Comm'n | 42 | 1,020 | 3 |
| 1929 | Wanaque | Belleville | Wanaque Aqueduct | N. J. D. W. S. Comm'n | N. J. D. W. S. Comm'n | 74 | 171,500 | 3 |
| 1929 | Belleville | Elizabeth | New'k.-Eliz. Line | Newark | Newark and Elizabeth | 60 | 25,400 | 3 |
| 1931 | In Newark | | Replaces Part of No. 1 | " | Newark | 42 | 2,255 | 2 |
| 1938 | Belleville | Newark | Part of No. 6 Line | " | " | 48 | 4,000 | 2 |

* Partly out of service since 1933.

TABLE 11

*Tests on 51-Inch Passaic Valley Water Commission Pipe*

Concrete Lock Joint pipe: 12-foot lengths; Nominal $D$ = 5.25 ft.; $D^{1.25}$ = 6.102; $D^{0.625}$ = 2.473

| TEST NO. | DATE | TIME | AGE OF PIPE | OBSERVATION RATING | LOCATION OF TEST SECTION STA. TO STA. | | $L$ LENGTH OF TEST SECTION | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = M'$ | $C_w$ WILLIAMS-HAZEN | $F_m$ MILLS | TEMP. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | Total | Per thousand feet $H$ | | | | |
| | | | *years* | | | | *feet* | *m.g.d.* | *feet per sec.* | | *feet* | *feet* | | | | °C. |
| P-1 | Sept. 12, 1932 | | 0.2 | | | | | | | | | | | 142.0 | | 18.4 |
| P-2 | Aug. 13, 1934 | | 2.1 | | | | | | | | | | | 136.0 | | 23 |
| | | | | | | | | | | | | | | 138.5 | | |
| | | | | | | | | | | | | | | 137.5 | | |
| P-3 | Mar. 24, 1939 | 2:05 P.M. | 6.7 | C | 2 + 60 | 72 + 94 | 7,034 | 56.4 | 6.15 | 37.82 | 13.52 | 1.922 | 0.0509 | 131.6 | 1.79 | 3.9 |
| | | 2:06 to 2:15 | | B | | | | 56.57 | 6.17 | 38.07 | 12.67 | 1.801 | .0474 | 136.7 | 1.85 | |
| | | 2:40 | | C | | | | 58.8 | 6.41 | 41.09 | 12.60 | 1.791 | .0437 | 142.5 | 1.93 | |
| | | 2:37 to 2:45 | | B | | | | 58.67 | 6.40 | 40.96 | 12.92 | 1.837 | .0448 | 140.3 | 1.90 | |
| P-4 | Mar. 24, 1939 | 2:05 | | B | 72 + 94 | 204 + 37 | 13,143 | 56.4 | 6.15 | 37.82 | 23.26 | 1.770 | .0540 | 137.6 | 1.86 | |
| | | 2:06 to 2:15 | | B | | | | 56.57 | 6.17 | 38.07 | 23.68 | 1.802 | .0474 | 134.9 | 1.85 | |
| | | 2:40 | | B | | | | 58.8 | 6.41 | 41.09 | 26.04 | 1.981 | .0483 | 136.6 | 1.84 | |
| | | 2:37 to 2:45 | | B | | | | 58.67 | 6.40 | 40.96 | 25.58 | 1.946 | .0476 | 135.9 | 1.85 | |

Construction was started in the summer of 1931 and the line was placed in operation July 4, 1932. Characteristics are as follows:

| ITEM | PER THOUSAND FEET OF PIPE | TOTAL |
|---|---|---|
| Horizontal curvature | 71° | 1,454°50′ |
| Number of horizontal curves | 2.57 | 53 |
| Net fall in line | 1.24 ft. | 25.6 ft. |
| Total vertical drop | 11.62 ft. | 239.0 ft. |
| Total vertical rise | 10.38 ft. | 213.4 ft. |
| Total vertical change in elevation | 22.00 ft. | 452.4 ft. |
| Number of vertical changes in grade | 4.57 | 94 |

It should be noted that the horizontal curvature is vastly greater than that in any of the other lines. Further, it is important to note that six of the curves are right angles, with four others having angles of 45°. Since these angles occur, for the most part, in streets, it is obvious that the radius of curvature must be short. Vertical curvature is important too. Four of the grades are in excess of 20 per cent and three more exceed 10 per cent.

Because of its thick concrete wall, the line requires fewer vacuum air valves than most steel lines. There are three air valve chambers, each having two 4-inch vacuum air valves and two having, in addition, pressure air valves. Also, there are four hand operated pressure air valves. The line has four 42-inch line valves with Venturi type approaches.

*Tests on 51-Inch Concrete Pipe.*

Tests on this pipe were made on September 12, 1932, August 13, 1934 and March 24, 1939. Those on the last date were made in conjunction with the interconnection survey and complete information on them is available. Adequate flows were obtained by starting the required number of pumping units at the upper end of the line and discharging the excess (over and above consumption) into a reservoir at the lower end. Volume of flow was measured by a Venturi meter at the upper end. The test section was between stations 2 + 60 and 204 + 37, a distance of 20,177 ft. or virtually the entire line. An intermediate observation was made at station 72 + 94. Results of the tests are given in Table 11.

TABLE 12

*Comparison of Construction Data of Six Conduits*

| | PEQUANNOCK CONDUITS | | ROCKAWAY CONDUITS | | WANAQUE AQUEDUCT | | DYER'S LINE, LITTLE FALLS TO PATERSON |
|---|---|---|---|---|---|---|---|
| | No. 1 | No. 2 | No. 1 | No. 2 | HASKELL TO LITTLE FALLS (TWIN LINES) | BALANCING RESERVOIR TO BELLEVILLE (SINGLE LINE) | |
| | Macopin to Pompton Notch | | Montclair to Jersey City | | | | |
| Date constructed | 1892 | 1896 | 1902 | 1921 | 1929 | 1929 | 1932 |
| Length of line, *ft.* | 28,300 | 28,300 | 56,707 | 56,700 | 65,178 | 23,537 | 20,542 |
| Diameter, *in.* | 48 | 48 | 72 | 72 | 74 | 74 | 51 |
| Length of sections, *ft.* | 7 | 7 | 7½ | 30 | 30 | 30 | 12 |
| Longitudinal joints | Riveted | Riveted | Riveted | Lock-bar | Lock-bar | Lock-bar | None |
| Girth joints | " | " | " | Riveted | Riveted | Riveted | Lock-joint |
| Material (predominating) | ¼" steel | ¼" steel | 7/16" steel | 7/16" steel | 7/16" steel | 7/16" steel | Concrete |
| Units per thousand feet: | | | | | | | |
| Degrees horizontal curvature | 21.7 | 21.7 | 10.2 | 17.3 | 24.6 | 12.3 | 71.0 |
| Total change in elevation—*ft.* | 37.3 | 37.3 | * | 18.6 | 6.3 | 22.4 | 22.0 |
| Vertical changes in grade | 3.40 | 3.40 | * | 1.76 | 1.59 | 3.48 | 4.57 |
| Air valve chambers | 0.71 | 0.71 | * | 0.49 | 0.51 | 0.72 | 0.15 |
| Blow-offs | 0.50 | 0.46 | * | 0.26 | 0.46 | 0.51 | 0.24 |
| Manholes | 0.85 | 0.85 | * | 1.10 | 0.63 | 0.97 | 1.02 |
| Valves | 0.035 | None | 0.177 | 0.177 | 0.077 | None | 0.244 |
| Diameter of line valves—*in.* | 48" | — | 48 | 48 | 48 | — | 42 |
| Design Velocity (av.) *ft. per sec.* | 6.2 | 3.2 | 2.75 | 2.75 | 2.6 | 3 | 5.5± |
| " " (max.) " " " | 9± | 5± | 4± | 4± | 4 | 5 | 6.5± |
| Original $C_w$ | 112 | 105 | 112± | 115± | 125 | 127 | 142 |
| Per cent loss in first 5 yr. | 12.5 | 8.5 | 18 | 9.5 | 1.6 | 1.6 | 3.5 |

* Similar to Rockaway Conduit No. 2.

## Comparison of Four Pipe Lines

To bring forcefully to the reader the general characteristics of the pipe lines thus far described, there is shown in Table 12, a comparative summary. To a large extent this table is self explanatory. It embodies some of the data given in Table 1 and in the text and also includes additional information. Table 12 demonstrates the variation in curvature, elevation, and coefficients of the various lines in the region.

## Quality of Water

All the waters carried by pipes hereinbefore described are derived from locations on the watershed of the Passaic River within a radius of a few miles. Fundamentally, therefore, the waters are quite similar in character.

Some idea of the nature of these waters may be obtained from Table 13, which shows not only analyses, but other physical data regarding the sources of supply. It is important to note that the Pequannock supply has a very small intake reservoir and that this water is subject to higher turbidities than any other unfiltered supply in the vicinity. (Table 13 results affected by distribution storage.)

There is no question but that inherent characteristics of the waters have played an important part in the past with respect to variations in coefficients. It also seems probable that newer linings are much less subject to the influence of quality, although the role played by sterilization may be of greater importance than is generally realized.

One notable instance of the effect of chlorine on pipe accretions has been given by Hale (7), who showed that accumulations, consisting largely of manganese, were found in varying amounts in the aqueduct of the Catskill supply for New York City—the accumulation being greatest near the intake at Ashokan Reservoir and decreasing with distance downstream. These deposits were greatly reduced and sloughed off when chlorination started in 1925, although previous to that time they had been increasing ever since the start of operations in 1915.

Similarly, it may be observed that chlorine was not introduced in the Pequannock lines until they had been in service for more than two decades and even then adequate chlorination, as we understand it today, was deferred for more than another decade. Ammoniation was introduced about 1930.

The chlorination of the Rockaway supply for Jersey City was the first major instance of chlorinated water in this country, but the supply and lines had been in use for about five years before treatment was begun in 1908. The careful control of chlorine in this supply (at least as far as normal bacterial life is concerned) during

TABLE 13

*Analyses and Other Data—Large Surface Supplies in Passaic River Basin*

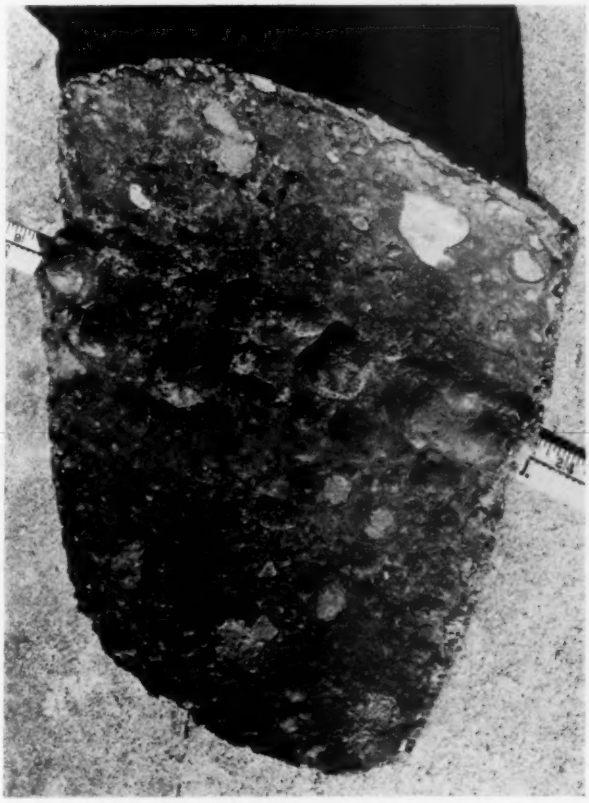
| | SOURCE NO. | | | | |
|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 |
| | Name of Source | | | | |
| | Pequannock | Rockaway | Wanaque (not filtered) | Passaic River (filtered) | Wanaque (filtered) |
| pH | 7.1 | 7.0 | 8.7 | 6.5 | 8.5 |
| Dissolved Oxygen, % *saturation* | 86.0 | | 81.8 | 92.8 | 86.1 |
| Micro-organisms, *standard units per ml* | 220 | 420 | 291 | 7 | 11 |
| Amorphous matter " " " " | 191 | 300 | 280 | 25 | 25 |
| Color—*p.p.m.* | 18.4 | 21 | 15 | 6 | 6 |
| Turbidity —*p.p.m.* | 1.25 | 3 | 3 | 1 | 1 |
| Alkalinity | 17.2 | 27 | 18 | 27 | 16 |
| Hardness (soap)—*p.p.m.* | 30 | 50 | 27 | 68 | 29 |
| Carbon dioxide—*p.p.m.* | 2 | 3 | 0.0 | 12 | 0.0 |
| Total solids—*p.p.m.* | 48.3 | 71 | 57.7 | 120 | 56 |
| Loss on ignition—*p.p.m.* | 18.0 | | 23.2 | | |
| Manganese—*p.p.m.* | * | trace | 0.11 | .06 | .02 |
| Iron—*p.p.m.* | 0.24 | — | 0.07 | .11 | .04 |
| Temperature, *degrees Centigrade* | 11.8 | 11 | 9.1 | 12.1 | 8.9 |
| Watershed area, *sq. mi.* | 64 | 119 | 94 | 760 | Same as |
| Storage, *total bil. gal.* | 12 | 8.5 | 29 | None | for |
| Storage, *available bil. gal.* | 11.2 | 8.5 | 27.6 | " | source |
| Storage ratio, *per cent* | 38 | 16.5 | 68.5 | " | No. 3 |

* Less than 0.1 p.p.m.

all the intervening years has no doubt had some effect on the carrying capacity of these pipe lines.

The water from the Wanaque supply suffered a bad case of red water trouble about six months after operation started. Correction of pH with lime was then begun (September, 1930) and this has, generally, been given complete credit for affecting a cessation of the difficulties. The writer believes that there is another factor,

having a vital effect, which seems to have been entirely overlooked. Coincident with increased pH, it was necessary to increase greatly the chlorine dosage to maintain a suitable bacterial reduction (a phenomenon frequently noted by various writers). The increased use of chlorine may easily have had its effect on reducing pipe line

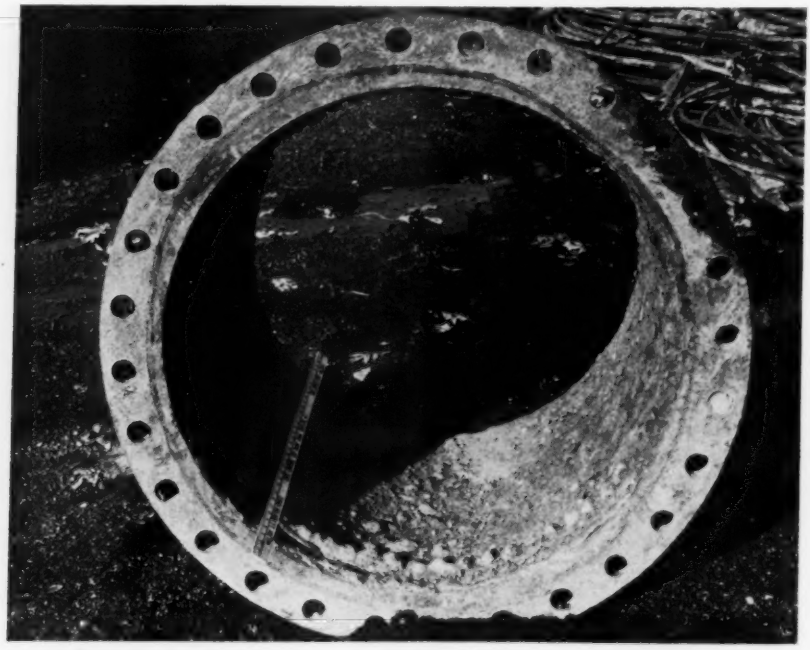


Atlantic City, N. J., 4-Inch Cast-Iron Distribution Main; laid 1884; removed 1940. Water considered highly corrosive

accumulations. The water is quite similar to the Catskill water of New York City and the process of pipe line cleansing noted by Hale (7) probably was duplicated, although in lesser degree, because of a very short period of operation prior to the change. As early as 1930, ammoniation was tried and, although it was then abandoned, the writer predicted its ultimate use and repeated the prediction in

1933. In the latter part of that year ammonia machines were installed. How much this influenced pipe line coatings will never be known, but it is certain that higher residuals are now employed and that there is undoubtedly some effect from these.

In 1934, the Passaic Valley Water Commission, objecting to the presence of manganese in the Wanaque unfiltered water, constructed additional pressure filters so that all the water taken from this source could be treated further. About a year later, operation of



Atlantic City, N. J., 30-Inch Steel Transmission Main, with flange attached; laid 1901; removed 1938. Water considered highly corrosive

the filters was begun and, thereafter, all Wanaque water taken by that commission was filtered. Whether manganese in suspension was sloughed off or not, and how much increased chlorination and ammoniation had to do with certain complaints, are somewhat moot questions.

Attmore E. Griffin has drawn the writer's attention to the fact that modern methods in chlorination have materially reduced the organic composition of impurities in water. The consequent in-

hibiting of biological growths and tuberculation may be very appreciable. Extensive research is said to be under way in exploring these phenomena. Certainly, experiences with chlorine and chloramines in pipe lines, as recorded in various technical writings, add strength to the validity of the suggestions regarding chlorination.

### Results and Methods Elsewhere

Because of the use of concrete lining for steel pipe, and the case of a concrete line in the area hereinbefore discussed, it becomes reasonable to compare results in preceding tables with observations elsewhere almost regardless of the type of interior lining. The list of references to be cited is by no means complete, but some important observations indicate the type of changes found by others.

In 1919, the Winnipeg Aqueduct, a concrete horseshoe-shaped grade line, was completed and tested by a color method. Adamson and Grant (8) state that the design value (Kutter's formula) was $n = 0.013$. In 1919, the actual value of $n$ was 0.0115, and in 1925, an electric salt velocity test gave $n = 0.012$. In terms of quantity, the design capacity was 91 m.g.d (Imp.). In 1919, the actual capacity was 111 m.g.d. (Imp.), while in 1925, it was 100 m.g.d. (Imp.).

Clark (9) has recorded the cleaning of 2,000 ft. of a 48-inch steel industrial line under 30 ft. of water. The $C_w$ before cleaning was 57 and after cleaning, 87.

As a study in the economics of pipe line design, the articles by Hinds (10) constitute an authoritative work of primary importance. Apparently, all pressure pipe and cut-and-cover sections were allotted a value of $n = 0.013$, while open canals were taken at $n = 0.014$. This in no way affects the usefulness of this classic contribution to pipe line design. It merely reflects the tendency to continue the use of $n$, without making it clear whether or not it is Kutter's $n$ (although it would normally be assumed to be such).

In this case, Hinds has obtained limiting velocities of the following order: grade conduit, 7.18 ft. per sec.; grade tunnel, 8.40 ft. per sec.; canal, 4.45 ft. per sec.; and siphons, 5.65 to 7.98 ft. per sec., depending on the location and shape. These velocities reflect the relatively low cost power available in the case cited. Loss in head becomes relatively less in importance. In New Jersey, velocities of 4 ft. per sec. are generally about the limit, largely because loss of head is important and power costs are high.

Harkness (11) has reported that pipes cement lined in Charleston, S. C., in 1921, now have a $C_w$ as high as 140, while unprotected pipes have a materially reduced capacity due to the corrosive nature of Charleston waters. The following table is given for pipes relined by the Tate process:

Value of $C_w$ (Williams-Hazen)

| DIAMETER OF PIPE | BEFORE LINING | AFTER LINING | |
|---|---|---|---|
| | | On Basis of Nominal Diameter | On Basis of Final Diameter |
| *inches* | | | |
| 8 | 53 | 112 | 140.5 |
| 8 | 51 | 128 | 147 |
| 6 | 50 | 117 | 151 |
| 4 | 36.7 | 88.7 | 151 |

These, of course, are all small sizes of pipe. Mr. Harkness further reported that in Sydney, Australia, approximately 1,000 mi. of pipe was cleaned and lined by the Tate process. Coefficients based on actual finished diameter were usually about 145 when new and about 140 after five years of use.

In commenting on this paper, James E. Gibson pointed out that the $C_w$ was only an approximation, as velocity had some effect.

In their years of experience, those engaged in cleaning water pipes have found repeated cases of mains which have lost a large percentage of capacity on account of many influences. That carrying capacity can be restored by cleaning is an established fact. The difficulty is that once the water works engineer depends on the restored capacity or coefficient he has to clean all vital parts of a system periodically. It is to be hoped, in the future, that large mains will be so treated or lined as to make periodic cleaning virtually unnecessary.

The value of longer lengths of pipe sections is now being appreciated. A 54-inch line being installed as an intake for Grand Rapids, Mich., is made of smooth bore pipe, each piece being 120 ft. long. It required three flat cars to carry one length. Compare this with the 7-foot lengths of the old New Jersey lines! The trend toward the use of pipes capable of producing continued high efficiency is unmistakable. There is definite need for a recapitulation of flow formula data and for a flexible arrangement that will meet the demands for each type of pipe.

The City of Denver, Colo., laid some 54-inch concrete pipe in 1921, with extensions in 1925 and again in 1939. Fred C. Scobey (12) reported in detail on tests on this pipe. In 1939, he, again, made tests on these lines. These latest results have been published in brief by the Lock Joint Pipe Company, which has supplied the writer with the data. Table 14 and Fig. 7 give a summary of all of these findings. The results are somewhat contradictory to the generally observed effects of age and velocity on the value of $C_w$. It must be noted, however, that the actual losses are rather small and do not permit the percentage accuracy normally obtainable. Scobey took all the precautions for refinement that could reasonably be asked, and it must be concluded that with such high coefficients it is difficult to discern any appreciable change in the pipe during the various periods of service.

On December 31, 1937, a new pipe line 48 in. in diameter was installed by the City of Toledo, Ohio. The line consisted of welded steel pipe in 40-foot sections, with Dresser couplings and $\frac{3}{16}$-inch cement-mortar lining applied by a centrifugal machine. The total length was 15,430 ft., and the alignment was fairly good.

Tests were made on a section of the pipe, 9,033 ft. long, by the Pitometer Company in August, 1938. The flow was 36.5 m.g.d. and velocities of 4.34 and 4.44 ft. per sec. were observed.

Respective values for the two tests were as follows: total loss, 9.4 and 9.2 ft.; loss per thousand feet, 1.04 and 1.02 ft.; $C_w$, 140 and 142 respectively. The coefficients are based on a nominal pipe diameter of 48 in. Actual finished diameter of the pipe averaged about 47.4 in.

Perhaps the outstanding deviation from the commonplace is also at Toledo, Ohio, where a steel pipe 72 in. in diameter is being jacked into a rock tunnel (under the Maumee River) and then surrounded by a concrete jacket. In this case there is apparently complete reliance on the permanent smoothness of the tar coating on the inside of the steel pipe!

While the lower limit of large diameter pipe lines has been suggested herein as 36 in., recognition should be given to the well recorded results on a 30-inch steel line in Pennsylvania, reported by Gannett and Carpenter (13). The horizontal curvature, vertical bends and manholes are not far different from some of the New Jersey conduits. Line valves are, however, full size—an important feature. The pipe was in 30-foot lengths with one longitudinal weld.

TABLE 14

*Tests on 30-, 54- and 60-Inch Denver Pipe Lines; Concrete, Lock Joint Pipe—12-foot Lengths*

| TEST NO. | CONDUIT NO. | REACH NO. | DATE | AGE OF PIPE | OBSERVATION RATING‡ | $L$ LENGTH OF TEST SECTION | DIAMETER | $Q$ FLOW | $V$ VELOCITY | $V^2$ | LOSS OF HEAD | | $\frac{H}{V^2} = M'$ | $C_w$ WILLIAMS-HAZEN | $F_m$ MILLS | TEMP. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | Total | Per thousand feet $H$ | | | | Air | Water |
| | | | | *years* | | *feet* | *inches* | *m.g.d.* | *feet per second* | | *feet* | *feet* | | | | °C. | °C. |
| D-1 | 10 | 1* | 1925 | 4 | B | 18,598 | 54 | 26.46 | 2.575 | 6.631 | 4.972 | 0.2673 | 0.0402 | 154 | 1.95 | 26 | 20 |
| D-2 | | | 1939 | 18 | B | | | 26.52 | 2.580 | 6.656 | 5.466 | .2939 | .0441 | 146 | 1.87 | 21 | 17 |
| D-3 | 10 | 2 + 3‡ | 1925 | 0.5 | B | 19,676 | 54 | 26.46 | 2.575 | 6.631 | 6.488 | .3297 | .0496 | 138 | 1.76 | 28 | 20 |
| D-4 | | | 1939 | 15 | B | | | 26.52 | 2.580 | 6.656 | 5.574 | .2833 | .0425 | 149.5 | 1.91 | 20 | 17 |
| D-5 | 10 | 1* | 1925 | 4 | A | 18,598 | 54 | 35.96 | 3.50 | 12.250 | 9.375 | .5041 | .0411 | 148 | 1.93 | 29 | 20 |
| D-6 | | | 1939 | 18 | A | | | 35.88 | 3.49 | 12.180 | 10.080 | .5420 | .0445 | 143 | 1.75 | 30 | 16 |
| D-7 | 10 | 2 + 3† | 1925 | 0.5 | A | 19,676 | 54 | 35.96 | 3.50 | 12.250 | 10.653 | .5414 | .0508 | 143 | 1.86 | 30 | 20 |
| D-8 | | | 1939 | 15 | A | | | 35.88 | 3.49 | 12.180 | 9.618 | .4888 | .0401 | 151 | 1.95 | 30 | 16 |
| D-9 | 3 | | 1939 | New | A | 22,350 | 30 | 12.72 | 4.01 | 16.080 | 29.307 | 1.311 | .0818 | 147 | 1.98 | 8 | 9 |
| D-10 | 13 | | 1939 | 9 | A | 36,854 | 60 | 50.34 | 3.965 | 15.722 | 21.039 | .5709 | .0363 | 147 | 1.93 | 11 | 14 |

* Reach 1 is nearly straight.

† Reach 2 + 3 is sinuous vertically and includes one 48-inch valve.

‡ Rating of B given in some instances because of small loss of head.

In some cases, lengths of pipe were welded together on the bank. The balance of the joints were made in the trench with Dresser couplings. Pipe coating was bitumastic enamel, $\frac{3}{32}$ in. thick. The pipe was laid in the summer and fall of 1932 and tested on March 26, 1933. The line carries untreated water from a reservoir to a coal company which uses it for washing anthracite coal. In brief, the results on a total length of 50,700 ft. were as follows:

Value of $C_w$

| LENGTH | $V = 0.680$ | $V = 1.280$ | $V = 2.573$ | $V = 3.147$ | $V = 3.8102$ |
|---|---|---|---|---|---|
| First 31,900 ft... | 123.9 | 135.8 | 144.6 | 143.6 | 143.5 |
| Second 18,800 ft.. | 145.5 | 144.5 | 147.3 | 143.5 | 145.6 |
| Total 50,700 ft... | 130.8 | 139.6 | 145.8 | 143.5 | 143.4 |

($V$ = Velocity in feet per second).

No doubt critics will hasten to point out that the results on the first 31,900 ft. of pipe dispute the writer's theory that the Williams-Hazen formula tends to give too high a coefficient at low velocities. Actually, a perusal of the records of this particular set of tests shows a net pressure drop in head of only 2.15 ft. at the lowest velocity for the first sectior and 0.94 ft. for the second. When it is noted that records were taken by pressure gages, reading in pounds, it is obvious that no great weight can be given to the low velocity results. For velocities over 2 ft. per sec., the records are all that could be asked. This tabulation bears out the contention that smooth bore steel pipe, well lined, can give initial results comparable to concrete pipe.

An indication of the degree of importance attached to proper pipe coating for small sizes of pipe was particularly noticeable in a paper by Billings and Brown (14), who drew attention to the fact that they had obtained a $C_w$ of 166 with a new type of coating on cast-iron pipe, whereas the ordinary coating had given 160. These results were obtained from a specially laid test section. The new coating consists of one rough coat and one final or smooth coat, the thickness being much less than ordinarily used and, therefore, giving a larger area for the same nominal diameter. In discussing this paper, Gibson declared that a section of pipe with the new type coating had been laid in Charleston, South Carolina, and showed a $C_w$ (under typical operating conditions) of 150, based on a calipered diameter.

Brush (15) pointed out that a rough pipe, to which coating would adhere better, had been in use in Brooklyn since 1863 and had out-

lasted many later pipes, retaining its original surface to a remarkable degree. In commenting on the paper by Billings and Brown, Brush said that the new coating, having been laid on a rough surface, would adhere better and show a longer effective life.

### Comparative Value of Various Formulas

It is only natural, in reviewing the findings developed by a study of this nature, that consideration should be given to the fundamental

Atlantic City, N. J., Close-up of Section of 30-Inch Steel Main, showing joint; laid 1901; removed 1938. Water considered highly corrosive, but tuberculation is far less than in less corrosive Pequannock water

theories of pipe line flow and that, if possible, a flow formula which will yield results close to actual experience should be selected. Apparently, no one has been able to dispute the Bernoulli theorem. Briefly given, it states that the sum of the gravity head, pressure head and velocity head is equal for all sections of a pipe line without friction loss. The theorem still holds true when friction occurs, but one side of the equation must contain an expression for the friction loss. In all practical problems, recognition of such a loss is of prime

importance, but the methods of estimating the loss are devious and sometimes incorrect.

In selecting notations to use in presenting data on hydraulics, there is much merit in employing terminology already in common use. Scobey (3, p. 3) has done this in excellent fashion and, in introducing new terms, has classified each in a thoroughly understandable manner. The following symbols are therefore briefed directly from Scobey, except for the friction factor, $F$, which will be described later. The value $M'$ differs slightly from Scobey's notation.

*Terminology:*

$d$ = Mean inside diameter of pipe in inches
$D$ = Mean inside diameter of pipe in feet
$Q$ = Mean discharge of pipe, in second-feet
$A$ = Mean area of pipe bore, in square feet
$L$ = Length of pipe or of reach tested, in feet
$h_f$ = Loss of head, in feet = $\frac{HL}{1{,}000}$
$H$ = Loss of head, in feet per thousand = $\frac{1000h_f}{L}$
$h_v$ = Velocity head
$P$ = Wetted perimeter, in feet = $2\pi r$ (for circular pipe)
$R$ = Hydraulic radius = $\frac{A}{P}$ or $\frac{D}{4}$ (for circular pipe)
$S$ = Hydraulic slope, in feet per foot of pipe = $\frac{h_f}{L}$
$C$ = Coefficient "of retardation"—by Chezy formula
$C_w$ = Coefficient "of retardation"—by Williams-Hazen formula
$K$ = Retardation coefficient, in equation of $H$ instead of $V$
$K_s$ = Retardation coefficient, as given by Scobey
$M$ = Retardation coefficient in any one pipe equation. It is the intercept on line $V = 1$ from logarithmic plotting of $H = MV^2$, as given in Fig. 14.
$M'$ = Intercept $M$, when projected from individual points at accepted slope of 2.0 (Scobey used 1.9). Used only where data are insufficient for development of $M$. (Note: $M' = \frac{H}{V^2}$)
$F_c$ = Retardation coefficient corresponding to Chezy $C$, except that $D$ and $H$ are substituted for $R$ and $S$
$F_w$ = Same for Williams-Hazen formula

$F_m$ = Same for Mills formula
$F_s$ = Same for Scobey formula

Basically, most hydraulic computations originally considered water as a falling body, whence came the term $\frac{V^2}{2G}$, or, as it is commonly called, velocity head. This resulted in attempting to express friction loss in terms of velocity head, a method that has much merit and is still useful for many purposes. Probably the most widely used equation of this type is that of Darcy:

$$h_f = f \frac{L}{D} \frac{V^2}{2G} \quad \text{.....(A)}$$

Church (16) has shown that such an equation may be supported by mathematical proof. Apparently the pioneer, Chezy, in 1775, recognized the suitability of such an expression and merely by rearranging the terms and substituting his notation, equation (A) above may be written in his original form:

$$V = C\sqrt{RS} \quad \text{.....(B)}$$

Unfortunately some authors have given the impression that formulas (A) and (B) are different.

For perhaps a half century or more after the Chezy formula was announced, it appears to have been accepted without great question, and indeed it has been of inestimable value. Then, in the latter part of the last century, there came a succession of formulas, generally similar to that of Chezy but differing slightly in exponents, that left hydraulics students gasping for breath. Probably many of these efforts were made because existing formulas did not agree closely with certain experimental data. Those engineers who distinctly labeled their formulas as applying to only certain types of pipe realized the limitations, but many apparently foisted their efforts on an unsuspecting world with little regard for the confusion that would be caused.

In more recent years, the experimental records of Mills, Saph and Schoder, and Scobey have been outstanding, and their deductions must be accorded a proper value. The writer has attempted to adapt their records to the problem at hand, aided by personal observations and such other records as were considered applicable, to the end that the following portion of this paper could be written.

For pure theory and exactness, the work of Mills (1) is one of the best contributions known to the writer. As reference books for methods of evaluating formulas, the works of Scobey (3, 17) are the clearest and most understandable yet found. Some repetition of his findings will necessarily be given here in order to bring out certain points, but wherever possible these quotations will be as brief as practicable. Anyone desiring to study the problem further is invited to read the original articles carefully as well as the very useful and informative handbook by King (18).

In order to present a comprehensive view of the formulas to be discussed herein, there is given the following tabulation:

| AUTHOR | ORIGINAL FORMULA | COMPARATIVE FORMULA IN TERMS OF V | COMPARATIVE FORMULA IN TERMS OF H |
|---|---|---|---|
| Chezy | $V = C\sqrt{RS}$ | $V = FD^{0.5}H^{0.5}$ | $H = K\dfrac{V^2}{D}$ |
| Williams-Hazen | $V = CR^{0.63}S^{0.54}0.001^{-0.04}$ | $V = FD^{0.63}H^{0.54}$ | $H = K\dfrac{V^{1.852}}{D^{1.167}}$ |
| Mills | $S = \dfrac{C_1V + C_2V^{2*}}{R^{1.25}}$ | $V = FD^{0.625}H^{0.5}$ | $H = K\dfrac{V^2}{D^{1.25}}$ |
| Scobey | $H = K\dfrac{V^{1.9}}{D^{1.1}}$ | $V = FD^{0.58}H^{0.526}$ | $H = K\dfrac{V^{1.9}}{D^{1.1}}$ |

* Approximate form is $S = \dfrac{CV^2}{R^{1.25}}$.

As mentioned before, Chezy found that an approximate formula could be written in the simple form shown, with the radical sign to express the powers of $R$ and $S$. Clark (19) has pointed out that once $R$ and $S$ were removed from the radical, they were given almost every exponent available. Scobey (3, p. 80) rearranged a large number of formulas in terms of $H$ and thereby indicated the relatively small difference between many of them. He seems to be the most insistent of the modern writers in discarding the cumbersome $S$ for the far more comprehensible $H$, and replacing the outmoded $R$ by $D$. Everyone who once uses $H$ and $D$ becomes impressed with the soundness of the change.

Scobey has emphasized the general form $H = K\dfrac{V^z}{D^x}$ in which $z + x$ approaches a value of 3.00, as first suggested by Reynolds more than a half century ago. This form is of particular interest in hydraulic

power development, where pressure is of prime importance. It has the distinct advantage of producing exponents that are almost universally greater than 1.0, therefore making solution more comprehensible to the average user. In the great majority of potable water supply problems, however, the desire is to deliver a quantity of water to meet maximum demand, and pressure is, more often than not, sacrificed to assure an adequate volume. For this reason, the writer greatly prefers the form of $V = F\,D^a\,H^b$. Of course the exponents $a$ and $b$ are fractional and therefore not so readily understandable. The coefficient $F$, however, is directly proportional to the volume—a most important feature in supplying water to myriads of consumers. The coefficient $F$ differs from $C$ only in that $D$ and $H$ are respectively 4 and 1,000 times as large as $R$ and $S$.

Regardless of which form is preferred, it is obvious that the four formulas, compared herein, differ only in relatively small increments in the exponents. The ideal formula, of course, would be one where a given coefficient would yield accurate results for all combinations of $V$, $D$ and $H$. No one has yet produced such a formula, and the writer is beginning to believe that no one ever will. Myriads of records substantiate this belief. Williams and Hazen tried to arrive at a fair average. They developed a formula that gave approximations over a wide range but did not come very close to fitting the conditions outlined in this paper.

It should be noted that Scobey (17) originally developed a formula identical with the approximate form of Mills, but proposed it for concrete pipe only, whereas Mills considered it applicable to all types of pipe. Apparently each of these writers arrived at the same formula independently. Even though Scobey published his first, it is believed that Mills had actually developed the basis for his findings many years before and for that reason it will be called the Mills formula, without intending in any way to reflect upon the excellent work of Scobey.

In order to obtain a graphic representation of these formulas, within the range of use for the pipes tested or for others proposed in the future in the same area, a pipe diameter of 48 in. was selected and assumed to be operating at a normal design velocity of 4 ft. per sec. These values were substituted in the Chezy formula with $C = 100$, and the resulting loss of head was obtained. With this slope or loss and the same $V$ and $D$, corresponding coefficients were determined for the Mills, Williams-Hazen, and Scobey formulas. Subse-

quently the coefficients thus found were employed with velocities varying from 1 to 10 ft. per sec., to compute the loss of head for each velocity and formula. Curves were plotted with $V$ and $H$ as ordinates. Of course, all curves obtained by this method intersect at a velocity of 4 ft. per sec. and while the coefficients are low, the results are relative.

The same procedure was followed for diameters varying from 3 to 10 ft. The results, plotted on logarithmic paper, are shown in Fig. 12. A little study will indicate, to anyone familiar with algebraic computations, why the Mills and Chezy formulas give identical results when the above method is used.

It is not contended that this graph demonstrates the superiority of one formula over another. It does show, however, that the Scobey formula gives results between the extremes of the others.

Williams and Hazen (20, p. 18) have suggested that the selection of a value for $C_w$ be started at a velocity of 3.25 ft. per sec., or approximately one meter per second. There is much merit in this suggestion. Conditions frequently encountered, however, involve a velocity of 4 ft. per sec. With this velocity, and a "large pipe" diameter of 4 ft., the form of equation suggested by Scobey may be reduced to $H = K (4)^{z-x}$. For the formulas mentioned herein, the value of $z - x$ becomes as follows:

Chezy..1.0 Mills..0.75 Williams-Hazen..0.685 Scobey..0.8

The Flamant formula, widely used at one time, gives 0.5. It seems obvious that under such conditions these formulas cannot all give parallel results over a wide range of velocities. In fact Scobey (3, p. 82) has pointed out that the exponent of $V$(*i.e.* $z$) may vary with different tests on the same pipe line.

Of course, the same comparison, given where $D$ and $V$ are equal, may be used for a diameter of 5 ft. and a velocity of 5 ft. per sec., or $D = 6$ and $V = 6$ and so on.

In order to indicate the probable superiority of the Mills formula (which is hereinafter chosen in its approximate form as the most adaptable to the problem at hand), Table 15 has been prepared from the excellent series of results in the first test of the No. 1 Rockaway conduit in 1903 (see also Fig. 8) together with the October, 1936, series on the relined portion of Pequannock conduit No. 2. The formulas used were rearranged in uniform fashion as shown in a previous paragraph. Here again, it is obvious that the Chezy and Mills formulas will produce variations of equal value. If data for

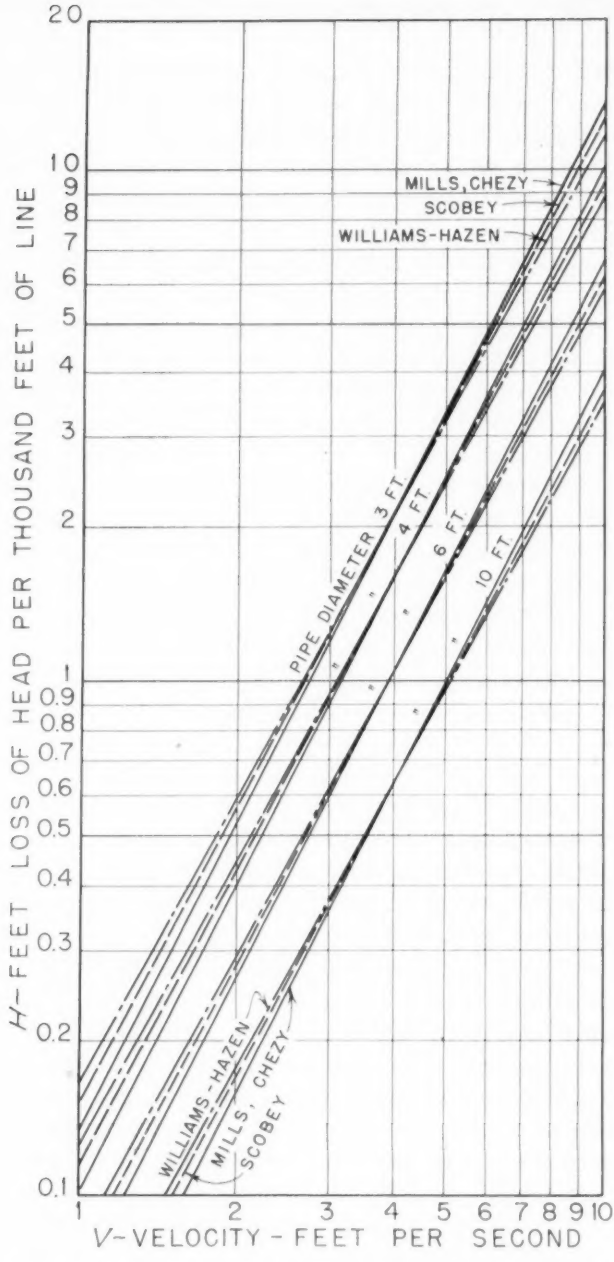



FIG. 12. Relation of $V$ to $H$ for Various Formulas; Assumed $V = 4$, $C = 100$, $S$ found (Chezy), then corresponding coefficients for other formulas. From this, remaining values were computed

the other curves in Fig. 8 were computed, generally similar results would be obtained, although in less striking fashion.

Still further comprehension of these conditions may be gathered from Fig. 13, showing the results of Table 15 graphically. The

TABLE 15

*Comparison of Various Formulas on Basis of Unit Power of Velocity; Form of equation is* $V = FD^x H^y$

| $V$ = VELOCITY FT. PER SEC. | $H$ = LOSS OF HEAD FT. PER THOUSAND | $F_c$ CHEZY | $F_w$ WILLIAMS-HAZEN | $F_m$ MILLS | $F_s$ SCOBEY |
|---|---|---|---|---|---|
| (For 1903 tests on Rockaway (Jersey City) Supply Main No. 1, 72-Inch Riveted Steel) | | | | | |
| 1.14 | 0.061 | 1.885 | 1.670 | 1.506 | 1.755 |
| 1.37 | .098 | 1.787 | 1.553 | 1.429 | 1.644 |
| 1.52 | .131 | 1.715 | 1.473 | 1.370 | 1.566 |
| 1.83 | .186 | 1.732 | 1.468 | 1.385 | 1.568 |
| 2.09 | .252 | 1.700 | 1.423 | 1.359 | 1.527 |
| 2.60 | .382 | 1.718 | 1.414 | 1.373 | 1.526 |
| 2.69 | .396 | 1.745 | 1.435 | 1.395 | 1.549 |
| 2.69 | .417 | 1.701 | 1.395 | 1.359 | 1.508 |
| 3.29 | .610 | 1.720 | 1.390 | 1.375 | 1.509 |
| 3.30 | .615 | 1.718 | 1.388 | 1.373 | 1.507 |
| Max. variation, *per cent.* | | 10.85 | 20.3 | 10.85 | 16.45 |
| (For Oct. 1936 Tests on Pequannock (Newark) Conduit No. 2,48-Inch Riveted Steel, Relined With Cement Mortar) | | | | | |
| 3.078 | 0.542 | 2.089 | 1.772 | 1.756 | 1.900 |
| 3.903 | 1.000 | 1.951 | 1.630 | 1.641 | 1.746 |
| " | 0.987 | 1.965 | 1.640 | 1.652 | 1.758 |
| 4.309 | 1.345 | 1.862 | 1.530 | 1.565 | 1.650 |
| " | 1.270 | 1.910 | 1.580 | 1.607 | 1.700 |
| 5.368 | 1.955 | 1.920 | 1.560 | 1.617 | 1.689 |
| Max. variation, *per cent.* | | 12.2 | 15.8 | 12.2 | 15.1 |

curves obtained from the Rockaway pipe line figures of 1903 would indicate that uniform relations between velocities and coefficients should be found when the velocity exceeds 2 ft. per sec. Mills consistently disregarded any results with velocities of less than 1.5 ft. per sec. He based his theory largely on the numerous tests on small brass pipes by Saph and Schoder and on more than 1,000 tests

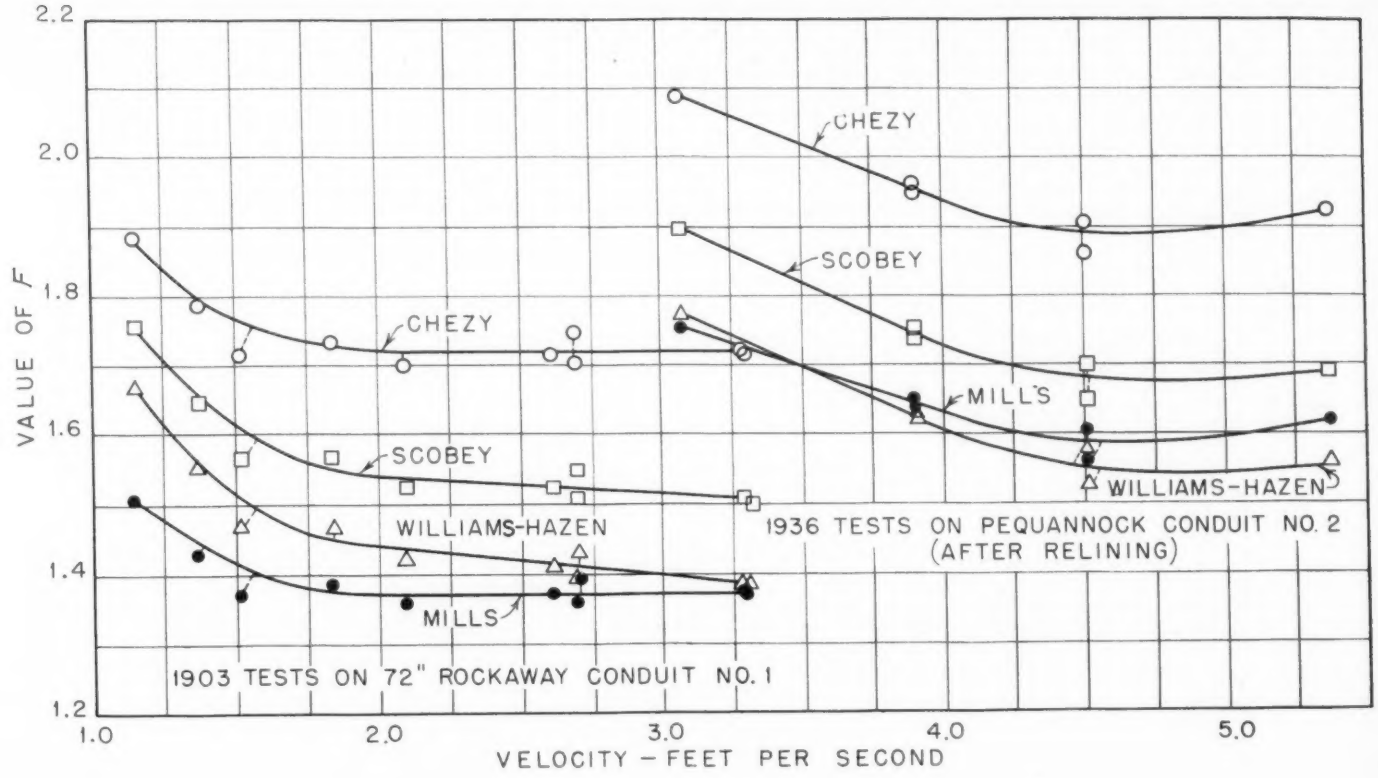



FIG. 13. Relation of $F$ to Velocity (Various Formulas); $F = \frac{V}{D^a H^b}$

of his own on 12-inch cast-iron pipe. In these, it was shown that uniform results could not be obtained in very small pipes with velocities lower than 6 ft. per sec., while for the 12-inch pipe the lower limit was 2 ft. per sec. Evidently, Mills deduced 1.5 ft. per sec. as a lower limit of dependability in dealing with large diameter pipes.

The general similarity between the curves for the 1903 tests on the 6-foot Rockaway pipe and the 1936 tests on the relined 4-foot Pequannock pipe, as shown in Fig. 13, is distinctly noticeable. There is indicated the possibility that the Chezy and Mills formulas give results that actually start to curve back again at higher velocities.

It would be presumptuous to declare, on the strength of the meager results shown in Fig. 13, that the superiority of the Mills formula is firmly established, particularly in comparison with the tremendous number of graphs shown by Scobey (3) in his Fig. 3. This much is established by the present writing—that the Mills formula is superior to others, in practically every case so far examined, for large pipes in northern New Jersey, and that the Chezy formula is equally good for individual tests. Later, it will be shown that the Chezy formula is not applicable over so wide a range because of the inaccuracy of the exponent of $D$.

*Exponent of $V$.*

To permit ease of comparison with Scobey's work, the formulas are to be considered in the form $H = K \frac{V^z}{D^z}$ in which Scobey prefers a value for $z$ of 1.9, whereas Mills and Chezy both give 2.0. Figure 14 has been prepared to show some of the typical tests on the Pequannock and Rockaway conduits, as published by both Mills and Scobey, as well as some of the later results. In this diagram the logarithmic relations of $V$ and $H$ are plotted. While the series by no means approaches the total number shown by Scobey (3) in his Fig. 3, it demonstrates clearly the conditions, existing in the pipes, which have been given particular emphasis in this paper. It seems clear that the exponent of $V$ is nearer 2.0 than 1.9 in the composite of these typical tests.

Scobey (3, p. 81) says that the exponent of $V$ should be 1.9 for full-riveted pipe and 1.95 for smooth bore or girth-riveted pipe. The definite trend toward smooth bore pipes, of far greater smoothness than was ordinarily predicted to be possible a few years ago, leads to the belief that the 1.9 value may be largely, if not wholly, disregarded,

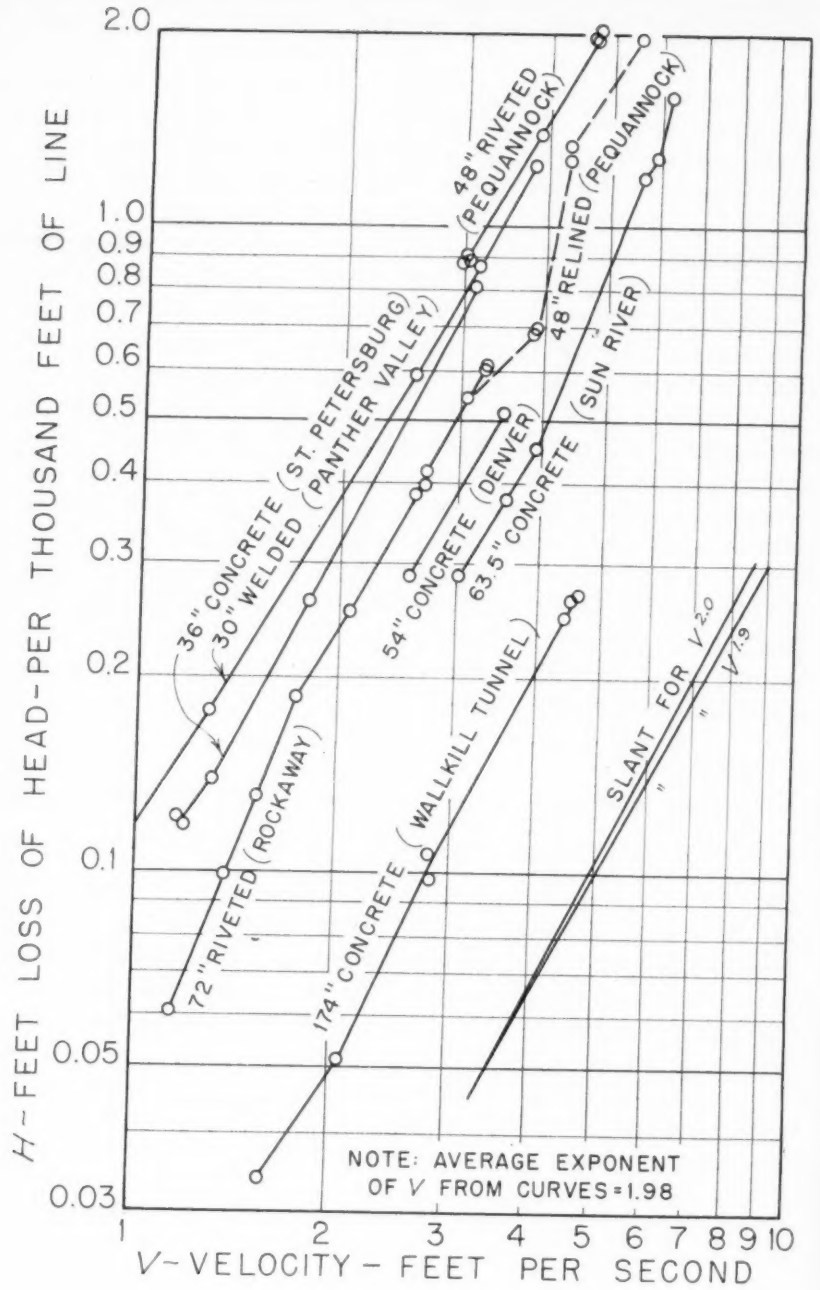



FIG. 14. Exponent of $V$, in $H = K \frac{V^n}{D^x}$

in future pipe design, in favor of a higher value. While the weight of Scobey's evidence may be in favor of using 1.95 or even 1.90, it still appears to the writer that any possible error involved in using 2.0 as the exponent of $V$ is far less than the original error in assuming a value for $C$, $K$, or $F$.

In further support of assuming the exponent of $V$ to be 2.0 instead of a lesser figure, the writer has made a careful study of the curves, tables and formulas presented by Scobey (3). Taking the computed exponents of $V$ for full-riveted pipe in sizes 36 in. in diameter and larger, it is found that the average figure is 1.91 for twenty-three series of tests, ten of which were made on the pipes previously described herein. For the seven series on the Rockaway conduit No. 1, the average is 1.92.

In Scobey's tabulation of girth-riveted pipes, the only series given, for diameters of 36 in. or more, shows an exponent of 2.08. Similarly, for continuous interior pipe, the average of three series is 1.99. For compound pipes of large diameter, the exponent is given for $Q$ (because $V$ is variable) as 2.12, 1.99 and 2.23, all these tests having been made on new pipe in 1921.

While from the standpoint of number of tests, the 1.9 value seems well substantiated, there is no question in the writer's mind that the pipes of today more nearly approach the conditions found in pipes of smoother character, wherein Scobey's results demonstrate the possibility of an exponent above 2.0. Furthermore, a study of Fig. 8 indicates that the 1908 tests of the Rockaway conduit No. 1 are "out of line." If this set is omitted, the average exponent of $V$ would become 1.94; and it should be noted that with one exception, the series would then be in a steadily descending order with increasing age, starting with 2.03 for nearly new pipe. Taking into consideration the fact that present day pipes tend to maintain their original values much more than formerly, it is apparent that a value of 2.0 is not unreasonable as the exponent for $V$.

Mills has so well substantiated his theory involving terms of both $V$ and $V^2$, that there is little room for doubt that it is the explanation for much of the variation found by so many authors. It suggests why so many exponents of $V$ are possible; but Mills, himself, advocated an approximation employing only $V^2$ in the final use of his formula. It is important, however, to note, again, that the bulk of his calculations were based on tests of small pipes and on one 12-inch pipe. A study of his curves indicates the possibility that uniform

results in large pipes cannot be obtained until velocities of at least 2 or 3 ft. per sec. are reached. If such is the case (many tests having been made at lesser velocities), there is no point in trying to obtain an exponent of $V$ to a fine degree of accuracy. Furthermore, Mills repeatedly showed that, at certain velocities in the lower range, the exponent of $V$ actually went much higher than 2.0 at times, but gradually receded to about that value as velocities increased.

*Exponent of D.*

Mills (1, p. 95) remarks, merely, that the logarithmic determination of the exponent of $D$, as based on numerous observations, showed him that a value of 1.25 was indicated. This appears to be a rather nonchalant dismissal of the subject without submitting proof, but anyone who really studies his book cannot help believing that Mills must have used the same infinite care in arriving at this conclusion that he employed in all of his deductions.

Scobey (3) in his Fig. 5 submits voluminous data to support his exponent of 1.1, but the wide spread of the points plotted permits a rather broad interpretation of results.

To arrive at a conclusion concerning the relative merits of various exponents, Fig. 15, in which the relation of diameter to $M$ and $M'$ is plotted on logarithmic paper for many of the typical results recorded herein, as well as a number of others largely taken from records of Mills and Scobey, has been prepared. From a study of the curves, it appears that almost any value between 1.2 and 1.25 may be selected as an exponent of $D$. Actually the one that seems to be most representative is 1.23. Scobey (3, p. 82) has pointed out that a rounded out value of exponents is all that is justifiable. Furthermore, there is no excuse for adding another exponent to an already overburdened field, merely for the sake of being different. For these reasons, the use of the Mills exponent of 1.25 is clearly indicated.

*Final Selection of Formula.*

The great weight of evidence encountered indicates that the approximate Mills formula, in the form of $V = F_m D^{0.625} H^{0.5}$, is the best to advocate for all around use in the design of modern large diameter pipes.

There is no intention to attempt, in any way, to discredit the extremely commendable work by Scobey. Rather, it is the writer's opinion that the later Scobey formulas are excellent, provided they

are limited in use to certain specific classes and sizes of pipe. In obtaining an exponent for $V$ of 1.9 for steel pipe, Scobey used tests on pipe varying from less than 4 in. up to 14 ft. in diameter. Naturally, it is difficult to produce a formula that will fit such a wide range. Mills attempted to meet this by simply changing the values of $C_1$ and $C_2$, in his formula, for each type and size of pipe, although he arrived at certain final approximations to fit most cases. He clearly indicated, however, that discrepancies of 10 per cent or more do occur.

Many engineers will undoubtedly wonder why the Williams-Hazen

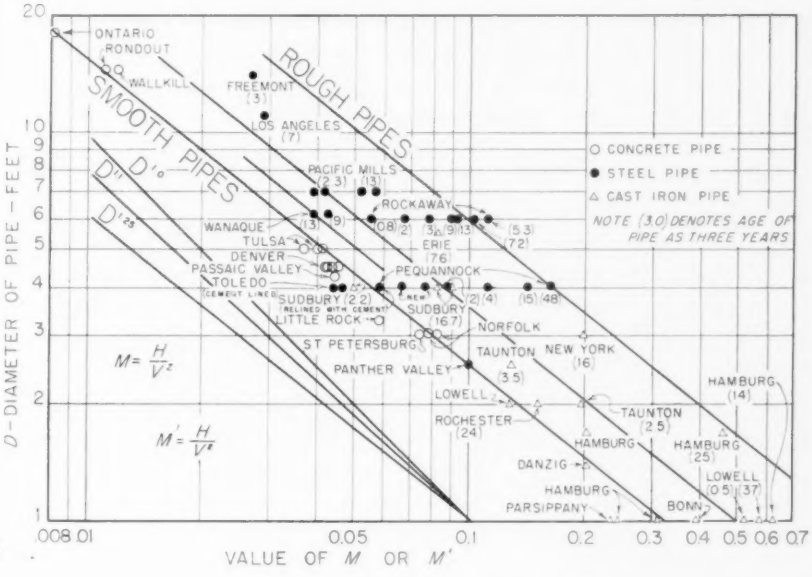



FIG. 15. Exponent of $D$, in $H = K \frac{V^2}{D^x}$

formula is not proposed. There are several reasons. In the first place, it is awkward to handle. Col. F. F. Longley has suggested that the principal reasons why the formula received such an enthusiastic reception were that it was based on a large number of experiments, thus insuring some confidence, and that the tables and slide rule made it easy to use. Since the slide rule is no longer on the market, and since the tables do not give data for some of the sizes recorded herein, the value of the formula is materially reduced. Furthermore, it falls far short of giving a reasonable result when low velocities are considered (see Fig. 8). Of course, these low

velocities are seldom critical in pipe line design, but it must be borne in mind that, within the range of ages and velocities shown in Fig. 8, the Williams-Hazen coefficient varies 20 per cent while the Mills coefficient varies only 10.85 per cent.

In the suggestions made by Williams and Hazen, Mills, and Scobey regarding decrease in carrying capacity with age, none of the conditions obtained by recent lining methods is even closely approximated. The New England Water Works Association Committee (4, p. 319) shows the Williams-Hazen reduction at 1 per cent per year for 10 years. Scobey (3, p. 90) shows a deterioration of $1\frac{1}{2}$ per cent a year, compounded. Mills (1, p. 222) gives $0.17\sqrt{t}$ (where $t$ is age in years) for locations near the reservoir and $0.12\sqrt{t}$ for distances of more than a mile from the reservoir (based largely on the Pequannock results), as the increase in the coefficient of retardation, *i.e.*, $H = K\frac{V^2}{D^{1.25}}(1 + 0.12\sqrt{t})$. For early years, the Mills formula correction gives high relative losses, decreasing in percentage with age.

It is true that none of these authors had available the data used in preparing this paper and that, on the basis of information known to them, they have arrived at some remarkably correct answers. The point to be stressed here is that *the art of lining pipes has changed and formulas or coefficients must be changed to correspond.*

The late Frank E. Winsor has been quoted as saying that it made little difference how a steel pipe line was originally constructed, since, ultimately, the effect of retarding factors was such that all types would be similar. This opinion was well substantiated by earlier pipe line records but it is not true of recent ones.

## Effect of Roughness

Almost universally, engineers, upon inspecting pipe lines, have expressed surprise that an apparently small accumulation of one type or another on the inside of a pipe can cause a great reduction in carrying capacity. An excellent illustration of the effect of tuberculation or nodulation has been given by Clark (21), who indicates that nodulation may reduce the effective area of a pipe by five times the height of the nodule.

Some further explanation may serve to show why this is so. Assume for the moment that a coating or growth of any kind inside of a pipe, instead of changing the coefficient (as is assumed in general practice), actually changes the effective diameter (or, if you will,

the hydraulic radius); then it is found that the formulas for velocity and quantity may be written as follows:

| *New Pipe* | *Tuberculated Pipe* |
|---|---|
| $V = F\,D^{0.625}H^{0.5}$ | $V = F\,D_1{}^{0.625}H^{0.5}$ |
| $Q = VA = \frac{\pi F}{4}\,D^{2.625}H^{0.5}$ | $Q = VA_1 = \frac{\pi F}{4}\,D_1{}^{2.625}H^{0.5}$ |

Thus, it is clear that, if $F$ and $H$ remain constant, the discharge varies as the 2.625 power of the diameter. It seems likely that this condition could be well substantiated by operating experiences if it were possible to measure accurately the effective restriction caused by tubercles, in the manner that Clark (21) has indicated. Summarizing, it may be stated that it is the effective diameter that really changes during the process of corrosion or tuberculation and that any attempt to prove a linear relation of $F$ to age and corrosion is doomed to failure. In practice, however, there must be maintained a uniform value of $D$ in calculations, and, for that reason, good judgement in selecting values of $F$ for various conditions is all that can be expected.

TABLE 16

*Comparison of $F_m$ with Other Coefficients; Velocity Assumed at 4 Ft. per Sec.*

| PIPE DIAMETER | FOR $F_m = 1.0$ | | FOR $F_m = 1.5$ | | FOR $F_m = 2.0$ | |
|---|---|---|---|---|---|---|
| | $C_w$ (W.-H.) | $n$ (Kutter) | $C_w$ (W.-H.) | $n$ (Kutter) | $C_w$ (W.-H.) | $n$ (Kutter) |
| *feet* | | | | | | |
| 2 | 70 | 0.0180 | 109 | 0.0128 | 148 | 0.0101 |
| 4 | 72 | .0192 | 111 | .0133 | 152 | .0103 |
| 6 | 73 | .0198 | 114 | .0135 | 155 | .0104 |
| 8 | 74 | .0202 | 115 | .0137 | 157 | .0104 |
| 10 | 75 | .0204 | 117 | .0137 | 159 | .0103 |
| 12 | 76 | .0205 | 118 | .0137 | 160 | .0103 |

## Comparison of $F_m$ with other Coefficients

To permit specific comparison of the coefficient $F_m$ with other coefficients in common use, Table 16 has been prepared. Because the ranges and general comparisons have been stated previously, it does not seem necessary to elaborate on the question. The table empha-

sizes the statement that values of F between 1.0 and 2.0 about cover the range generally found in practice. This greatly simplifies correlation of various values used in design or developed by test.

### Nominal Diameter

Throughout the text, except where otherwise noted, it has been assumed that, where a pipe was classified as having a nominal diameter of 48 in., all calculations should be based on this figure. Table 2 shows clearly the type of variations that may be expected. On small pipes, many engineers like to put calipers on the finished inside and measure the true diameter, basing all calculations thereon. This, of course, is the proper method if truly comparative results are to be obtained.

With large diameters, linings, unless they are very thick, are less important. With the present trend toward smooth bore pipe, it seems that, with very little effort, pipe diameters can be made so that the finished inside diameter will be an even size, such as 4 ft. Certainly, most calculations are made on such a basis and manufacturers can probably arrange to meet such a situation. One manufacturer of small pipe has already announced that his pipe has been made slightly oversize to compensate for the thickness of the lining.

### Temperature

Little has been said about temperature. Probably Scobey (3) has given more thought and constructive suggestion to this than anyone else. His deductions are very interesting and instructive, regardless of the fact that some authors may not agree with those findings. That temperature and viscosity are linked, there can be little question. Proof is still lacking, however, regarding the effect of temperature on the pipe itself. Until such an obstacle is over-come, there seems little reason to go too far in the direction of correcting for temperature. Because of this, no calculations have been made to take its effects into account. Nevertheless, temperature records of water have been included where available.

### Diagram and Table for Use of Mills Formula

For the benefit of those who may wish to use the formula suggested herein, Table 17 includes the values of $D$ to the 0.625 and 1.25 power, which may be used, together with an ordinary slide rule, to solve the formula in terms of either $V$ or $H$. Figure 16 gives a diagram for the

graphic solution of the formula. This corresponds to the one given by Scobey (17, p. 54) except that Fig. 16 is based on $D$ (in feet) while Scobey's is based on $d$ (in inches). The use of $D$ is preferred because the coefficient $F_m$ then becomes greater than unity for most practical problems instead of always being fractional, as is the case when $d$ is employed. Figure 16 is somewhat unorthodox in shape, but it is believed that it may be employed readily and that it will provide a sufficient range of values for most problems.

No effort has been made to adapt this diagram to use for small pipes, although indications are that it is fully as useful and as accurate as those obtained from other formulas. If acceptance of the formula warrants, another diagram will be drawn covering an expanded range for small pipes.

## Conclusions

1 Riveted steel pipes have shown serious loss in carrying capacity in the past.

2. Much of this loss has been caused by disintegration of the pipe coating.

3. Improvements in making smooth bore pipes have increased pipe capacities.

4. Modern methods of making and applying coatings, both bituminous and cement-mortar, have greatly improved carrying capacities.

5. Horizontal bends in pipe lines are important, but sharp vertical curvature may be even more effective in reducing capacity.

6. Line valves should be as large as practicable where loss of head is important.

7. Age of pipe lines is far less important on modern pipes, of demonstrated durability of coating or interior surface, than formerly, in the effect on carrying capacity. Experience in New Jersey and elsewhere indicates that cement or concrete surfaces may improve with age.

8. Old pipes may be relined and restored to a capacity above the original in many cases, and may be expected to continue to deliver water at the increased rate.

9. Relining is an economic factor that cannot be ignored.

10. Quality of water has had much to do with reduction in capacity, but need not cause great concern in pipes having reasonably permanent interior surfaces.

## TABLE 17

*Values for Use of Mills Formula*

Values of $D^{0.625}$ (or $D^{\frac{5}{8}}$) for use in formula $V = F_m D^{0.625} H^{0.5}$

and " " $D^{1.25}$ (or $D^{\frac{5}{4}}$) " " " " $H = \dfrac{V^2}{F_m{}^2 D^{1.25}}$

| DIAMETER | | $D^{0.625}$ | $D^{1.25}$ | AREA $A$ | DIAMETER | | $D^{0.625}$ | $D^{1.25}$ | AREA $A$ | DIAMETER | | $D^{0.625}$ | $D^{1.25}$ | AREA $A$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $d$ | $D$ | | | | $d$ | $D$ | | | | $d$ | $D$ | | | |
| *in.* | *ft.* | *ft.* | *ft.* | *sq. ft.* | *in.* | *ft.* | *ft.* | *ft.* | *sq. ft.* | *in.* | *ft.* | *ft.* | *ft.* | *sq. ft.* |
| | | | | | 46 | 3.833 | 2.316 | 5.363 | 11.54 | 90 | 7.500 | 3.523 | 12.411 | 44.18 |
| 12 | 1.000 | 1.000 | 1.000 | 0.785 | 47 | 3.917 | 2.347 | 5.510 | 12.05 | 92 | 7.667 | 3.572 | 12.758 | 46.16 |
| 13 | 1.083 | 1.051 | 1.105 | .922 | 48 | 4.000 | 2.378 | 5.657 | 12.57 | 94 | 7.833 | 3.620 | 13.104 | 48.19 |
| 14 | 1.167 | 1.101 | 1.213 | 1.069 | 49 | 4.083 | 2.409 | 5.804 | 13.10 | 96 | 8.000 | 3.668 | 13.454 | 50.27 |
| 15 | 1.250 | 1.150 | 1.322 | 1.227 | 50 | 4.167 | 2.440 | 5.953 | 13.64 | 102 | 8.500 | 3.810 | 14.514 | 56.74 |
| 16 | 1.333 | 1.197 | 1.432 | 1.396 | 51 | 4.250 | 2.470 | 6.102 | 14.19 | 108 | 9.000 | 3.948 | 15.588 | 63.62 |
| 17 | 1.417 | 1.243 | 1.546 | 1.576 | 52 | 4.333 | 2.500 | 6.252 | 14.75 | 114 | 9.500 | 4.084 | 16.679 | 70.88 |
| 18 | 1.500 | 1.288 | 1.660 | 1.767 | 53 | 4.417 | 2.531 | 6.403 | 15.32 | 120 | 10.000 | 4.217 | 17.783 | 78.54 |
| 19 | 1.583 | 1.333 | 1.776 | 1.969 | 54 | 4.500 | 2.560 | 6.554 | 15.90 | 126 | 10.500 | 4.348 | 18.901 | 86.59 |
| 20 | 1.667 | 1.376 | 1.894 | 2.182 | 55 | 4.583 | 2.590 | 6.706 | 16.50 | 132 | 11.000 | 4.476 | 20.033 | 95.03 |
| 21 | 1.750 | 1.419 | 2.013 | 2.405 | 56 | 4.667 | 2.619 | 6.860 | 17.01 | 138 | 11.500 | 4.602 | 21.178 | 103.9 |
| 22 | 1.833 | 1.461 | 2.133 | 2.640 | 57 | 4.750 | 2.648 | 7.013 | 17.72 | 144 | 12.000 | 4.726 | 22.335 | 113.1 |
| 23 | 1.917 | 1.502 | 2.256 | 2.885 | 58 | 4.833 | 2.677 | 7.168 | 18.35 | 150 | 12.500 | 4.848 | 23.504 | 122.7 |
| 24 | 2.000 | 1.542 | 2.378 | 3.142 | 59 | 4.917 | 2.706 | 7.322 | 18.99 | 156 | 13.000 | 4.968 | 24.684 | 132.7 |
| 25 | 2.083 | 1.582 | 2.502 | 3.409 | 60 | 5.000 | 2.734 | 7.477 | 19.63 | 162 | 13.500 | 5.087 | 25.877 | 143.1 |
| 26 | 2.167 | 1.621 | 2.629 | 3.69 | 61 | 5.083 | 2.763 | 7.632 | 20.29 | 168 | 14.000 | 5.204 | 27.081 | 153.9 |
| 27 | 2.250 | 1.660 | 2.756 | 3.98 | 62 | 5.167 | 2.791 | 7.790 | 20.97 | 174 | 14.500 | 5.319 | 28.296 | 165.1 |
| 28 | 2.333 | 1.698 | 2.883 | 4.28 | 63 | 5.250 | 2.819 | 7.947 | 21.65 | 180 | 15.000 | 5.433 | 29.520 | 176.7 |
| 29 | 2.417 | 1.736 | 3.014 | 4.59 | 64 | 5.333 | 2.847 | 8.105 | 22.34 | 186 | 15.500 | 5.546 | 30.755 | 188.7 |
| 30 | 2.500 | 1.773 | 3.144 | 4.91 | 65 | 5.417 | 2.875 | 8.264 | 23.04 | 192 | 16.000 | 5.657 | 32.001 | 201.1 |
| 31 | 2.583 | 1.810 | 3.275 | 5.24 | 66 | 5.500 | 2.902 | 8.423 | 23.76 | 198 | 16.500 | 5.767 | 33.255 | 213.8 |
| 32 | 2.667 | 1.846 | 3.408 | 5.58 | 67 | 5.583 | 2.929 | 8.582 | 24.48 | 204 | 17.000 | 5.875 | 34.519 | 227.0 |
| 33 | 2.750 | 1.882 | 3.541 | 5.94 | 68 | 5.667 | 2.957 | 8.743 | 25.22 | 210 | 17.500 | 5.983 | 35.793 | 240.5 |
| 34 | 2.833 | 1.917 | 3.676 | 6.30 | 69 | 5.750 | 2.984 | 8.904 | 25.97 | 216 | 18.000 | 6.089 | 37.075 | 254.5 |
| 35 | 2.917 | 1.952 | 3.812 | 6.68 | 70 | 5.833 | 3.011 | 9.065 | 26.74 | 222 | 18.500 | 6.194 | 38.367 | 268.8 |
| 36 | 3.000 | 1.987 | 3.948 | 7.07 | 71 | 5.917 | 3.038 | 9.228 | 27.50 | 228 | 19.000 | 6.298 | 39.668 | 283.5 |
| 37 | 3.083 | 2.021 | 4.085 | 7.47 | 72 | 6.000 | 3.064 | 9.390 | 28.27 | 234 | 19.500 | 6.401 | 40.977 | 298.6 |
| 38 | 3.167 | 2.055 | 4.225 | 7.88 | 74 | 6.167 | 3.117 | 9.718 | 29.87 | 240 | 20.000 | 6.503 | 42.295 | 314.2 |
| 39 | 3.250 | 2.089 | 4.364 | 8.30 | 76 | 6.333 | 3.170 | 10.047 | 31.50 | 246 | 20.500 | 6.605 | 43.619 | 330.1 |
| 40 | 3.333 | 2.122 | 4.504 | 8.73 | 78 | 6.500 | 3.222 | 10.379 | 33.18 | 252 | 21.000 | 6.705 | 44.955 | 346.4 |
| 41 | 3.417 | 2.155 | 4.646 | 9.17 | 80 | 6.667 | 3.273 | 10.713 | 34.91 | 258 | 21.500 | 6.804 | 46.298 | 363.1 |
| 42 | 3.500 | 2.188 | 4.787 | 9.62 | 82 | 6.833 | 3.324 | 11.047 | 36.67 | 264 | 22.000 | 6.903 | 47.645 | 380.1 |
| 43 | 3.583 | 2.220 | 4.930 | 10.08 | 84 | 7.000 | 3.374 | 11.386 | 38.48 | 276 | 23.000 | 7.097 | 50.369 | 415.5 |
| 44 | 3.667 | 2.253 | 5.074 | 10.56 | 86 | 7.167 | 3.424 | 11.727 | 40.34 | 288 | 24.000 | 7.288 | 53.120 | 452.4 |
| 45 | 3.750 | 2.284 | 5.218 | 11.04 | 88 | 7.333 | 3.474 | 12.067 | 42.24 | 300 | 25.000 | 7.477 | 55.901 | 490.9 |

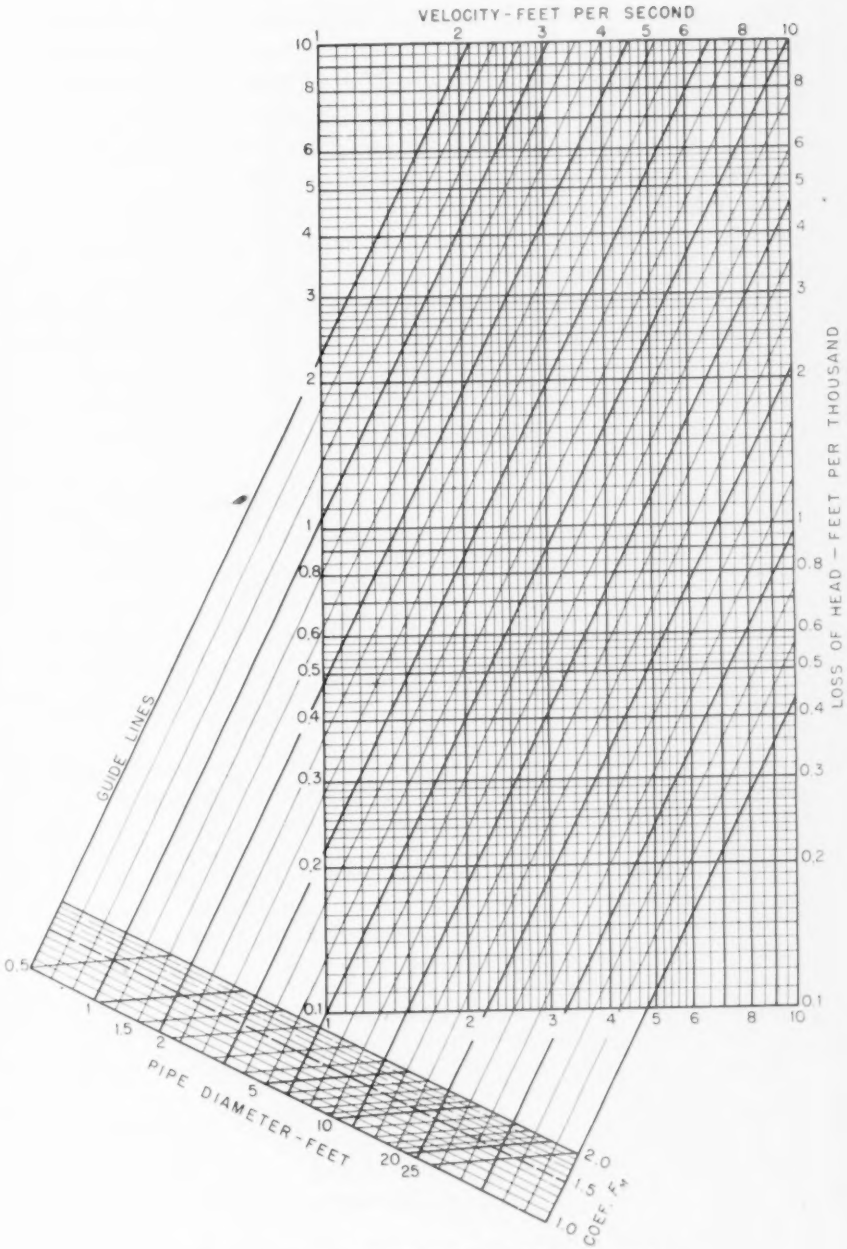



FIG. 16. Diagram for Solution of Mills Formula

11. Concrete pipe is little affected by quality of water, except insofar as the carrying of minerals into solution by very aggressive waters is concerned. The same is true of steel pipe with cement-mortar linings.

12. Higher chlorine residuals probably have an important bearing on inhibiting growths and accumulations in pipe lines.

13. While the Chezy formula has much to recommend it, particularly in its simplicity of handling, careful studies indicate that it does not conform to data now available.

14. Studies described in this paper lead to the recommendation of the modified Mills formula in the form $V = F_m D^{0.625} H^{0.5}$, (*i.e.* $D^{\frac{5}{8}} H^{\frac{1}{2}}$). The terms are as follows:

$V$ = velocity in feet per second.
$F_m$ = friction factor or coefficient.
$D$ = diameter in feet
$H$ = loss of head in feet per thousand feet of pipe. This may also be written $H = \dfrac{V^2}{(F_m)^2 D^{1.25}}$

15. The use of this formula simplifies rather than complicates calculations. This is because $H$ is easier to use than the old term $S$; $D$ is more comprehensible than $R$; and $F_m$ is a figure easily remembered.

16. Normal variations of $F_m$ are between 1.0 for an old tuberculated pipe and 2.0 for a smooth pipe of fairly straight alignment.

17. The following table expresses the values of $F_m$ for various conditions:

| *Value of $F_m$* | *Type of Pipe* |
|---|---|
| 2.0 | Smooth pipe of fairly straight alignment |
| 1.8 | Smooth pipe, sinuous alignment |
| 1.6 | Fairly smooth pipe, average alignment |
| 1.4 | Fairly smooth pipe, sinuous alignment |
| 1.2 | Tuberculated pipe or rough pipe |
| 1.0 | Pipe greatly deteriorated in capacity and probably requiring relining. |

(Note: many values may be found below 1.0, but in those cases the pipe is generally in need of rehabilitation.)

## Acknowledgements

Cooperation of the New Jersey State Water Policy Commission, George S. Burgess, Chairman; and the North Jersey District Water

Supply Commission, Pearce R. Franklin, Chairman; together with the Federal Works Projects Administration, made possible the interconnection survey from which this paper was developed. The writer had direct charge of the engineering work, under the general guidance of Mr. Burgess and in cooperation with Howard T. Critchlow, Engineer-in-Charge, State Water Policy Commission. Ralph E. Lowe was State Supervisor for the W.P.A. on this project. Engineers representing the State Water Policy Commission were Russell S. Wise and B. A. Furman, the latter having been particularly helpful in bringing the work for this paper to a conclusion.

Staffs of the two above mentioned Commissions, as well as those of the Passaic Valley Water Commission and Division of Water of the City of Newark aided materially. The data regarding the Pequannock conduits, as furnished by the Newark engineers, have been most useful.

Many other water engineers and officials, as well as representatives of underwriters and manufacturers, have furnished information and assistance, and their cooperation is herein gratefully acknowledged.

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# The Water Superintendent and the Elected Official

*By Frank D. Livermore*

*Illustrated by H. W. Montague*

Superintendent, Type 1—who gets on the payroll through purely political methods

I HAVE a faint suspicion that the real reason for the presentation of this paper is to bring out some point which may assist you either to get a job or, if you have one, to remain on the payroll with a minimum of friction and a maximum opportunity for more pay. In the writing, no punches have been pulled, but if anything rather too blunt has been said, it was tossed out with the full knowledge that it will bounce back just as quickly.

To clarify his own relation to the subject, the writer wishes to point out that he is a licensed municipal engineer—not an elected official—who often goes from one meeting, where he sits as a member of the governing body, to another, where he is just a borough engineer, taking orders instead of giving them, and appreciating both positions. It is in his experience in this work that the writer has formed the opinions which are presented here.

All water works superintendents are divided into three groups—at least as far as methods of attaining office are concerned. The first group, for whom I have little respect, gets on the payroll through purely political methods. I believe you will agree with me that we should devote a little time to a consideration of them and of the

A paper presented on October 19, 1940, at the New Jersey Section Meeting, Atlantic City, N. J., by Frank D. Livermore, Commissioner of Public Works, Ridgewood, N. J.

elected officials who put them into office, since both are a disgrace to all of us. We all have a vital stake in the problem, since the incompetency of such appointees certainly reflects on the profession as a whole.

I am of the opinion that we might well consider the appointment of a special committee to deal with such cases, so that, wherever they are known to exist, we may, through newspaper releases and similar publicity, put the problem before the public, with particular reference to its ultimate cost to the taxpayer. Most newspapers are glad to print material of this sort, and, if it comes from authoritative sources, it carries with it much more weight than you probably imagine. The light of publicity does wonders to many ugly things which thrive in the dark.

Another group is made up of men with whom any self respecting skunk would refuse to associate. I refer to those who seek to get a job by cornering officials and making behind-the-back slurs on the ability of the men whose jobs they want, at the same time praising their own abilities to such an extent that they would have us believe that Saint Peter has retained them to remodel Heaven's own water system. As far as I am concerned, they should be given leaky buckets and set to bailing out the River Styx. Though I have never met a superintendent of this type, without a doubt there are some. Such a way of getting a job is not only reprehensible but poor technique, since any worthy public official should be able to see through it in a minute, and, taking it for what it is, would hold in utter contempt the author of the attempt.

The remaining group of water works men—those who are honest, capable and sincere in their desire to do a good job—constitute more than 90 per cent of the total. These are men whose height of ambition is to operate a municipal system as efficiently as if it were privately owned. That may sound impossible, but it can be done. Perfection in that aim may not often be reached, but if the superintendent and the public official have sense enough to take the time to get each other's slant on the subject, the job is half done.

In looking back over thirty years of experience as an appointed engineer and nearly ten years as an elected official, I have come to the conclusion that too many engineers and too many officials look at each other through a set of plain lenses when they need bifocals. The two groups arrive at their offices through directly opposite techniques and so cannot judge each other in the light of their own experience.

Post-Election Portrait of the New Commissioner

As an engineer you got where you are because you know your job and because you have spent practically your whole life eating, sleeping and talking water. You may have come up the hard way, with a minimum of technical education; on the other hand, you may be a college graduate who was a cub engineer for some corporation before you landed where you are now. In any event, you are certain that you know your business and you hate like blazes to have an outsider tell you how to run it. Right at this point, let me say that you do not have to, in most cases, if you will take time enough to acquaint yourself with your "Chairman of the Water Committee" or "Public Works Commissioner." Most of them are regular fellows and just as anxious to do a real public service as you. The big difference is in the manner whereby they are chosen. Unless you everlastingly and eternally keep that one point firmly fixed in your minds, this discussion is all wasted.

It may be that your boss wanted to run for public office, but the odds are that some friends or a political party picked him to run. Put yourselves in his position for a while and see what has happened. First, he probably went through a primary campaign and won. Certainly he went through a general election and won. After the election, the newspapers and his friends told him what a great fellow he was and what was expected of him. Then, in the course of events, the Mayor tossed him overboard, literally, into the water department to sink or swim. He may have been a good custom tailor, an insurance salesman, a grocery clerk, or what have you—even a municipal engineer—but, so far as you are concerned, he is all the "Dumb Doras" of the world rolled into one. If you are testing a new well and are dumb enough yourself to start talking about "wire to water," he probably thinks that is some new contraption whereby you fellows, when at conventions, can call up some mermaid for a heavy date, and lets it go at that. The chances are that in his own line, he is just as smart as you are in yours, and that, above all,

he really wants to be a good public official. He may come into office with a slightly exaggerated idea of his ability and importance, but if you will think back to the time when you were made superintendent, you may recall that you too probably had to buy a new hat. Probably it soon came down over your ears, particularly on the day when your new main supply line blew a few joints on wash day and the women had finished telling you just what they thought. Your public official will soon get his bumps and deflate to normal, so don't worry too much about that.

I know of no infallible technique to follow in all cases, but in nine out of ten, I recommend this: Don't wait for him to come to you. Call on your new director at your very earliest opportunity, not with your hat and tin cup in hand, nor with that "Gosh, has this got to be done," attitude, but with the air of a man who is sure of himself and who has a department of which he is proud. Then, introduce the new boss to *His* department. I have emphasized the word "his," because in that word is all the meat of the nut. You are really the fellow who gets all the dirty work of running the plant, but don't make the fatal mistake of making the new boss think you regard him as just the tail of the dog—a necessary evil—and that you are going to do as you please, regardless.

The Commissioner's Vision of "Wire-to-Water"

If your plant is what it ought to be (and if it isn't, it's your own fault) an inspection trip will amaze the average man and if you can convince him that this is *his* baby to bring up and to brag about, while you wash the bottles and other things, you have little to worry about. He may know little or nothing about the technical details of what he has seen, but don't kid yourselves into thinking that the average man can't size up a general situation and the men who control it in very short order.

If you have a program of expansion or replacement which you know ought to be done, don't just go ahead and do it. First, take time enough to tell the "Commissioner" what you have in mind,

"WE"—the Inception of a Mutual Admiration Society

ask him what he thinks about it, and above all, say "we" when you talk about it. Colonel Lindbergh made his biggest hit in Paris when he said "We" flew the Atlantic. If you can make your commissioner feel that it is "we" who are doing a good job of running the water department, you are sitting pretty. You catch more flies with molasses than with vinegar and because it's the fly who signs your pay check.—Don't forget it! No elected official, with any sense, wants to be smeared with a lot of soft soap, but your own actions should convince him that his water superintendent knows his business and, at the same time, respects the fact that, in the last analysis, it is the elected official who is directly responsible to the public and must hold the bag.

Speaking of holding the bag, you can do much to make your job more secure by remembering that it is you and your men who are the contact between the "Commissioner" and the public. Nothing worries an official so much as having to spend time getting out of a peck of trouble into which some fresh meter reader or collector has gotten him. This may seem trifling to some of you, but, take it from me, the boss does not soon forget these things and too many bulls pulled by lazy or impertinent subordinates may get you into hot water. If you can drum into your men that molasses and vinegar idea, it may make a tough dose to swallow when dished out by some unreasonable citizen, but, remember, it is better to take it that way, as a tonic, than as a purgative which will move them or you right out of a job.

Along the line of this "we" business, here is a tip—everyone likes to see his name in print and every reporter likes a good or novel story for his paper. No one could or should criticize a superintendent for wanting to get favorable publicity for some special job he has done, particularly if it is of the unusual variety; but did it ever occur to you to analyze the feelings of the "Commissioner" when he picks up the paper and learns from it, rather than from you, what has been going on in the department. It puts him in a very bad light and the shadow may fall your way. It really is much

simpler to tell him before releasing the story; and, if the paper says: "Superintendent Joe Doakes, with the wholehearted cooperation of Commissioner Waterankle, has just done thus and so to make our water department even better than it was before, etc.," the boss will be more than ready to give you the credit which you really deserve. What may be even more important, when you hit him for a raise, you will find him much more approachable if he feels you are working *with*, rather than *in spite of* him.

To climb over to the other side of the fence for a moment, many elected officials have no more conception of how to get along with people, in general, and water superintendents, in particular, than does a fellow who passes out oranges at a Dublin fair. For such as these, my apologies to you. If this state of affairs comes from pure ignorance, perhaps you can do something about it—that is, if you have the patience of Job—but it is a well nigh hopeless task. My only suggestion is that you stick closely to your knitting and give no valid reason for his finding fault with your work.

Making the Job $worthwhile$

One other species, and I rate him lower than the fellow with the leaky bucket, is the bird who gets into office with the idea of chiseling wherever he can and who expects you to help him. To your credit, I know that this idea is repulsive to a real water man. Sometimes, though, for instance when the boss wants you to permit some favored contractor to lay a main with three feet of cover instead of four, when the last foot is rock, or when he wants you to purchase inferior goods on which he gets a cut, the problem is a knotty one.

In my experience, I have only met one such person and the method I followed at that time may interest you. I had condemned certain material which was patently below par, and then had received peremptory orders to allow its use for no other reason than that the contractor was a good fellow. Arranging to meet my superior and his boy friend at a certain time on the job, and taking a witness with me, I told the superior in question that either the inferior material

was to be removed forthwith or my resignation would be given to the papers the following day and that said resignation would set forth all the details leading up to it. I said something in a previous paragraph about people liking to see their name in print, but, believe me, if there is one thing a small-time shady politician fears, it is a newspaper which prints the truth. The stuff was removed from the job and both of us got along much better after that.

To the public official, I have two suggestions which I believe will go a long way toward helping the "we" business previously mentioned.

First and foremost, for the love of Mike, let your superintendent really run the plant. You and he should have a thorough understanding and accord as to policies and general methods of procedure, but from there on, give him the responsibility of detail and, above all, back him up when he has the courage and ability to do his work well. I know of nothing which destroys efficiency and morale more quickly than to require a competent man to run to his superior every time he needs to buy a couple of pipe nipples, or to find out whose service line he should lay first. If, for a good reason, you do not have confidence in the ability and honesty of your water superintendent to do these things, fire him and get some one else. Water superintendents have long since outgrown three-cornered garments. They are grown men and should be regarded as such. I firmly believe that a good one increases in value in direct proportion to the responsibility and cooperation which he receives from the public official.

The second suggestion pertains to those men who come into office imbued with the idea that all public employees are there for the sole purpose of extracting as much as they can from the public payroll and who don't care a hoot about giving anything in return. Above all, this type of official is very sure that conventions, in general, were invented by Satan himself and that the sole accomplishment during them is the installation of elevators in Hades. With this attitude, I disagree sharply. Frankly, when your superintendent wants to go to a convention, give him the jolt of his career by saying, not only, "yes," but that you will go along to see what goes on. It may cramp both your styles slightly, but I wager that it will prove an eye-opener for you, and, what is far more important, will make your man feel that he has a regular human being working *with* him rather than *over* him. The results will astonish you.

# The Superintendent of Tomorrow

***By H. V. Pedersen***

*This stimulating discussion of the superintendents' problems and the relation of the A. W. W. A. to the water works superintendent should be read carefully by every member. It may be suggested that the use of Association standards had not appeared to others, as it appears to Mr. Pedersen, to make possible, in itself, the employment of men who are not otherwise competent to carry on their work. That cities do, from time to time and in various places, entrust these important public services to men who are less than adequately trained, is a regrettable fact, but many students of affairs think it is less, rather than more, frequently true and some feel that the growing strength, membership and services of the A. W. W. A. are contributing a great deal to this improvement.*

*If the old phrase—"In union there is strength"—has any validity, every new member of the Association benefits himself as well as every other member by the part he plays in the affairs of the Association. Certainly, no one can object to exploring new fields of usefulness for the A. W. W. A. and, in this light, the suggestions made by Mr. Pedersen bear study and appraisal.*

A POSSE of small boys dashed from a vacant lot, out across a busy street, their wooden guns spitting imaginary bullets after a gang of retreating bank robbers, oblivious to the fact that several approaching cars were compelled to stop to avoid an accident. When admonished for their carelessness and asked why they did not spend a part of their time, occasionally, building a water works system or some such worthy project, instead of always being cowboys battling with Indians, or G-Men in pursuit of bandits, one of the gang indignantly retorted, "Aw, who wants to be a water works man?"

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A paper presented on November 14, 1939, at the Missouri Valley Section Meeting, Omaha, Neb., by H. V. Pedersen, General Manager of Water Works, Marshalltown, Iowa.

Now, it is not so much what this boy of eleven actually said, but the tone of utter disgust in which it was delivered that should cause all of us to sit up and take notice. Surely we can appreciate the fact that it is more thrilling to a small boy to mow down the enemies of society with machine guns just as many movies depict, or so many radio programs describe today, but why the positive expression of disgust at the mere mention of a water works?

Relating this incident to several groups of men and women on different occasions, it was observed that the sympathies seemed to be always with the boys. Even though the expressions were more controlled and tempered with mature sophistication, nevertheless there was that something which unmistakably inferred, "Well, just why should anyone play games pretending to be a water works man?" Not that average persons have a personal grudge against a water works man, nor do they lack a certain pride in their community's welfare, but their attitude in regard to the value of such a common thing as water, borders on contempt.

Water works engineers during the past 200 years have made truly wonderful progress in the design and construction of water works systems. A well designed and operated water works system is dependable and efficient. Engineers have developed pumping equipment to supply a continuous flow of water under many and rapidly changing conditions. Engineers have developed a wonderful method of transporting water which, compared to all other forms of transportation, is very economical. What other method of transportation would attempt to deliver water to all parts of a community for less than 3 cents per ton?

In populated areas, either rural or urban, there is very little surface water fit for human consumption. Men trained in the science of water treatment have developed methods of removing filth from water and making it safe for drinking purposes. If it were not for the discoveries of these chemists and bacteriologists and the ability of engineers to design practical plants for the treatment of water as well as the willingness of a few understanding citizens to shoulder the financial responsibility of building water purification plants, men might still be dying off by the thousands each year as a result of water-borne diseases. But, the funny part of it is that the general public today wants uninterrupted service and all the clean, wholesome water it can use without paying the proper price for it. What is still less understandable: let something happen to interrupt

service or the quality of the water regardless of whose fault it might be and the public begins to howl.

Usually it is the superintendent who is the target of most of the public's wrath. Usually it is the superintendent who must bear the brunt when the public complains about a water bill which he has made out in accordance with an established rate, based on the amount of money required to retire a debt and pay the cost of operation. It is the superintendent who is blamed when any portion of a distribution system is closed for repairing valves or cutting in new extensions. Even when an extended drought forces some water departments to place limitations on the amount of water used, or causes the water to become very difficult to treat, it is the superintendent who is told about it in no uncertain terms.

## Public Relations Problem

Many ideas have been advanced for correcting such an attitude and bringing about better public relations. Probably, if all the people were compelled to read all the articles in the technical journals produced for the benefit of the water works profession, the attitude of the public might be changed so that in time the water works engineer and water works superintendent would be given full credit for what they have accomplished. But, just try to make them. It has been suggested that newspapers be used to enlighten the public, but where it can be shown that one newspaper might co-operate in a campaign of public schooling, it can be shown that many newspapers hesitate. Newspapers as a rule do not consider it good business to devote space telling about the merits of a water works, or to say a good word about anyone connected with one. But let any little thing happen to interfere with water service or the quality of the water, regardless of who is to blame, and it is spread all over the front page. It has been suggested that a water works should publish and distribute its own literature. But other than for a yearly report, the public seems to resent spending public money for this purpose. Probably if a water department should elect to distribute any amount of literature without definite public approval, the public would consider it as being "propaganda." That might defeat the very purpose for which it was intended. The use of moving pictures as an educational medium seems to have exceptional merit and although a comparatively new idea, it is one worthy of careful consideration.

The American Water Works Association has, since its founding,

done a world of good for the water works profession. Members of this organization have worked hard to bring about a standardization of many things used in the construction of a water works. Everything has become so standardized that all one needs do is order an article approved by the A.W.W.A. and he will obtain the best that there is. But what has the A.W.W.A. done in regards to the attitude of the public towards water works systems in general, or to that group of public servants entrusted with the responsibility of supplying safe water? Is this phase of the water works business such an impregnable wall that the Association dare not match its wits against it. Or has it not been considered important to raise the standard of the so-called water works profession in the eyes of the public, or to establish the true value of the commodity in which it deals?

Speaking about this so-called water works profession, just what does it mean, and whom does it include? Has anyone ever attempted to define the term so that every one might understand it? Undoubtedly a technically trained engineer who has devoted years to the design of water works systems could be classified as a professional water works engineer. A chemist who has devoted years developing better methods of purification might be classified as a professional water chemist. Trained men employed by manufacturers to design the various articles that are used to make up a water works system might also be included as belonging to a so-called water works profession, but what about the water works superintendent, or the man who is in responsible charge of the operation and maintenance?

It is often thought that the person in responsible charge of a water works system need not be a technically trained man. This may or may not be as it should, but the important fact is that there are wide variations in the qualifications of water works superintendents throughout the country. The standard of qualifications required by a community for its water department staff reflects to a large extent the attitude the public has concerning the value of water. Men elected to city councils, as a rule, reflect the attitude of the public. They often do not realize the importance of a water works system, nor do they appreciate the true value of water. They are often elected to office on an economy platform. In order to carry out their pre-election promises, they seek to reduce the salary scale as much as possible, not realizing that this form of economizing is the most costly. When a good man refuses to work for the wages offered,

the Council is likely to appoint some one who will work for such salary, irrespective of the appointee's qualifications or experience. It is true that such a man may through diligent study get by after a fashion until he learns his business or he may even be replaced by someone else who will take the job for less than he is getting. This way of doing things is sometimes called an American Community's inalienable right. But does this practice raise the standard of the water works profession, or is the water works superintendent really a part of a profession? It appears questionable whether very many people in many communities, because of their attitude of contempt for the value of water, consider the superintendent, or any other part of a water works system for that matter, as being associated in any way with a profession.

### Raising Personnel Standards

All water works men should take cognizance of the future, analyze trends and counteract unsound tendencies. Everyone recognizes the good that standardization of methods and water works materials has done for the water works industry. But can standardization be talked about too much unless we are willing to include water works personnel? Already some cities and towns, because of too much standardization talk, appear to have reduced the qualifications of the man in whom they place the responsibility of looking after the water supply and water works property, to a level of a push button artist. The idea seems to be getting around that because water works practices have become highly standardized, any water works system can more or less look after itself.

Could a tendency of lowering superintendent qualifications in time seriously affect the membership in the A.W.W.A.? The Association seems to be always striving for new and more members. It has been stated that out of all those eligible to membership in the U. S., only a small percentage actually belong. Although many reasons have been advanced to explain this lack of interest in the Association, the most logical one probably is that those who do not belong believe they would receive no personal benefits. They probably feel that they have nothing to gain. If you add to the number who believe this way as a result of lowering of qualifications, membership in the A.W.W.A. will automatically decrease. Sooner or later a choice must be made between a policy on one hand of ignoring the great majority of water works superintendents and continuing an Assoc-

iation as a clearing house between the consulting engineer and the manufacturer, or a policy on the other hand of increasing the membership of the Association many fold by actually doing something which will tend to raise the standard of the average superintendent.

A little serious thought should also be given to the situation which compels state health departments to conduct schools of instruction for training the yearly turnover in water works personnel in order to protect the water supplies of the state with a reasonable degree of assurance. If every one is agreed that the majority of our communities are best served by training the person entrusted with the water supply *after having been appointed* to office, then the entire water works industry had better forget the A.W.W.A. for a while and concentrate on making this method of instruction as effective as possible. If, on the other hand it is agreed that it is better to require candidates for the office of water works superintendent to meet certain qualifications based on training and experience received *prior to employment*, then the A.W.W.A. and the entire water works industry, for its own good, should express this opinion in no uncertain terms.

The water works superintendent of tomorrow is going to be what the water works industry of today thinks he should be. It is hoped he will not become a swivel chair warmer or a push button artist whose peak of knowledge is that water flows out of a faucet from a system of pipes because the inside of a pump was made to spin round and round as a result of pushing a button. It is hoped that he will not become a mere puppet incapable of making decisions and afraid to make improvements without permission of a small group of highly trained specialists. It is hoped that as time goes on he will become a more important cog in the great water works machinery, understanding the basic principles of design, treatment of water and good management. It is hoped the A.W.W.A. will some day undertake a program of directing the attention of the people to the importance of a well designed and well managed water works system and will reorganize so as to attract a greater number of superintendents. It is hoped that the youngsters of tomorrow will not have the same attitude of contempt for the value of water and of public services as those mentioned at the beginning of this paper, but will grow up with such respect for man built water works systems that they will spontaneously imitate those whose duty it is to supply and protect the most precious thing to man on earth.

# Effective Management Under Municipal Ownership

*By L. A. Smith*

A MUNICIPALLY owned water utility should be operated in such manner as to be of the best possible service to the citizens of the community which it serves. This purpose involves the furnishing of an adequate supply of pure water at proper pressure for both general service and fire protection; the proper design and maintenance of both the pumping equipment and the distribution system, so that continuous service may be given with a minimum of interruptions; and the management of the utility in the most efficient manner, so that the water rates will be reasonable and the finances of the department adequate.

In this paper, the author does not propose to discuss the relative merits of municipal ownership and private ownership at any great length. Municipal ownership does, however, offer certain definite advantages, which must be realized, and certain inherent disadvantages, which must be guarded against if the utility is to be of maximum service to its community.

Among the advantages of a municipally owned utility are that its obligations, income, real estate and personal property are all tax exempt, that it is not subject to many of the Federal and state regulations which apply to private utilities, and that the rate of interest on its bonds, largely due to tax exemptions, is lower than that on the bonds of private enterprise. Its property tax exemption, however, works to the definite disadvantage of the city, itself, unless the utility pays taxes to the city at the local tax rate.

Among the disadvantages of a municipal utility are that its management is not, generally, as continuous as that under private ownership and that, in some cases, its employees are selected on the basis of political preferment rather than on merit. These conditions,

A paper presented on October 15, 1940, at the Wisconsin Section Meeting, Manitowoc, Wis., by L. A. Smith, Superintendent of Water Works, Madison, Wis.

however, are, in great part, being eliminated through the adoption of municipal civil service.

A municipally owned utility should not be operated in the same way as the other city departments, which derive their sole revenue from taxes, but rather as a private utility which necessarily must meet its entire budget from revenue. The relationship between the management of the water utility and the city administration should be cooperative, since both bodies are involved in the expenditure of public funds—even though some of the funds are paid as taxes and some are paid as water rentals.

### Wisconsin Law on Use of Funds

The statutes of the state of Wisconsin clearly define the method of using the income of public utilities owned by a municipality. Section 66.06, Paragraph 11-c provides that:

"The income of a public utility, owned by a municipality, shall first be used to meet operation, maintenance, depreciation, interest, and sinking fund requirements, local and school tax equivalents, additions and improvements, and other necessary disbursements or indebtedness. Income in excess of these requirements may be used to purchase and hold interest-bearing bonds, issued for the acquisition of the utility, or bonds issued by the United States or by any municipal corporation of this state, or insurance upon the life of an officer or manager of such utility, or may be paid into the general fund."

It will be noted from this provision that income is to be used not only to meet annual expenses, but to provide for additions or improvements; and that income in excess of these requirements may be invested in securities or in insurance upon the life of an officer or manager of such utility or may be put into the general fund. In some cases, in Wisconsin, a council, being hard pressed for funds, has used water department funds in violation of its legal rights. Obviously, this is unfair to the water consumers, as the service will certainly suffer in the long run. On the other hand, it is a fair opinion that any income, in excess of the physical requirements, should be paid into the general fund and thus help reduce the tax rate. One of the best methods of controlling the situation is to have a separate water fund from which appropriations can be made only by the Board of Water Commissioners.

## Management in Madison

After its construction in 1881, the Madison water plant was operated for several years by a council committee—a rather awkward method of procedure. In 1885, a water works commission, consisting of five members, three of whom were citizens elected by the Common Council, one, the mayor, and one, an alderman elected by the Common Council, was organized to manage the plant. The citizen members serve staggered terms of three years. The mayor is an ex-officio member, serving during the time he holds office; and the one councilman, elected annually by the Council, is also an ex-officio member. Since no provision in the state laws permitted such a commission, the State Legislature enacted an enabling clause for the purpose, and this clause was added to the special charter of the city. Since then, the board has operated continuously, and management under it has proved very satisfactory.

Annually, the Board of Water Commissioners submits a budget to the Common Council just as do the other city departments; and, almost without exception, it is approved unanimously. The budget provides for the payment of city taxes, interest on the city equity at 4½ per cent, and such purchase of city equity as seems desirable. Also, under income, it provides for adequate charges against the city for fire protection service, for water rentals for bubbling fountains and various city buildings, and for street sprinkling and flushing, storm and sanitary sewer flushing and city construction work. All public structures in the city, including churches, schools, fire stations, park buildings and fountains are metered, and the city pays for water service on the same basis as do all the other consumers.

The 1939 financial statement of the water department includes the following record of transactions with the city:

| *Income* | | *Expenditures* | |
|---|---|---|---|
| Fire protection | $69,426.00 | Taxes | $46,172.82 |
| Municipal bldgs., etc. | 16,631.46 | Interest on equity | 76,465.30 |
| Street sprinkling, etc. | 3,955.75 | Repurchase of equity | 22,500.00 |
| Steam for heating | 16,553.43 | Office rent | 2,000.00 |
| Total | $106,566.64 | Total | $147,138.12 |

In addition to these major transfers of funds, the city and the water department pay each other for all special services rendered between them.

## Details of Operation

The details of operation under this commission form of management demonstrate the efficiency that can be maintained under municipal ownership. The working schedule begins at 8:00 A.M. every day when the water department superintendent meets the assistant superintendent, the service foreman and the stockkeeper at the service building. At that time, the work for the ensuing day is laid out and all requisitions for materials, after being checked by the assistant superintendent, are presented for the signature of the superintendent. At 8:30, the superintendent has a conference, at the main pumping station, with the chief engineer, the assistant superintendent and the station stockkeeper. The stockkeeper presents a card showing the pumpage for the preceding day, the total pumpage to that date for the current year and the previous year, and the costs of generating steam and of pumping. Requisitions are also presented for approval at that time.

At 9:00 A.M., the superintendent goes to his office in the City Hall, taking with him the approved requisitions. Purchase orders, which are signed by the superintendent and the city auditor, are then issued and mailed directly to the vendor. In general, three methods of purchase are used:

1. On large items, such as pipe, coal and supplies of that type, contracts are let on an annual basis. The city purchasing agent takes the bids and makes the analysis and tabulations on construction items, and the Board of Water Commissioners generally takes bids on operating items, the main one being coal.

2. On many materials, such as small pipe and fittings, gasoline, oil, etc., purchases are made on a standard discount basis, on bids taken by the purchasing agent.

3. On small items which do not fall in either of these classifications, the stockkeeper, either at the service building or at the pumping station, gets bids from at least three vendors and issues the purchase order to the one who provides either the most advantageous price or the best delivery, depending upon the particular requirements.

The city treasurer is the treasurer for the water department, and all monies collected by the department are paid directly to him. Water consumers pay bills directly to the treasurer and, in the case of special items, pay-in vouchers are issued by the water department. All of the money collected by the treasurer for the department is deposited daily in a separate water fund.

The Board of Water Commissioners meets once a month, and at that time approves the payroll and all bills. These accounts payable are then recommended to the Common Council for allowance. The bills pass through the hands of the claims committee and are approved by the Common Council. Checks are prepared by the water department, to be signed by the mayor, the city clerk, and the city auditor.

### Financing of Improvements

In recent years, practically all improvements have been financed from current water works funds, but it was the procedure, formerly, to issue general city liability bonds for water works improvements. Prior to the last ten years, general city liability bonds bore an interest rate of about 4 per cent, whereas water revenue bonds bore a rate of 5 per cent. The water department and the city agreed that the financing should be done through the agency of general city liability bonds, the water department to pay interest at the rate of $4\frac{1}{2}$ per cent. This meant that both the city and the water department saved $\frac{1}{2}$ per cent by the cooperative procedure. Since 1930, only one improvement has necessitated financing by a bond issue, and in that case, water revenue bonds were issued directly by the water department. These have since been re-financed so that the present rate of interest is about 2 per cent. By reason of the fact that general city liability bonds were issued for water works improvements, and that some direct cash appropriations were made by the city to the water department, the city equity in the water works is $1,750,000, or half of the total property and plant valuation of $3,500,000. The water department pays $4\frac{1}{2}$ per cent interest on this city equity and, in addition, retires as great a portion as it can each year from available funds not needed for operation, maintenance or additional improvements to the plant. In the last six years, approximately $250,000 in city equity has been retired. Depreciation was formerly set up at a composite rate of $1\frac{1}{5}$ per cent. This has recently been revised so that the composite rate for operating depreciation is approximately $1\frac{1}{2}$ per cent. The depreciation reserve is not built up in cash, but is invested either in improvements to the plant or in the repayment of city equity. This seems to be a sounder method of procedure from the standpoint of the water department, since new money can be obtained at 2 per cent, whereas the unpaid city equity bears a rate of $4\frac{1}{2}$ per cent.

### Selection of Employees

All water department employees come under the jurisdiction of the Board of Personnel of the city and enjoy civil service status. When additional employees are required, application is made to the Board of Personnel. The Board submits a list of eligible applicants from which the new employee is selected. If, however, no applicant is satisfactory to the department, additional names of eligibles may be requested. Employees so engaged serve a probation period of six months, after which they are either released or retained as permanent employees. It is possible for the superintendent, or any other department head, to engage temporary help without civil service classification, if the men are employed for a period not to exceed three months.

The management procedure outlined in the foregoing pages has been in effect for many years and appears to be satisfactory in every respect. It is not necessarily recommended for adoption by other cities, but it does deserve consideration by all who are concerned with the management of municipal utilities. Conditions vary and local experience may indicate, in many cases, that the system which has been in use for many years is more desirable, provided, of course, that the impediments to its effectiveness are incidental and not such as to require a wholesale reorganization.

# What Water Softening Would Do for Wisconsin Cities

*By Louis R. Howson*

SOFTENING of the public water supplies of Wisconsin offers worthwhile savings. It is the purpose of this paper to show the importance of softening of the public water supplies to the communities of the state and to estimate the savings that would result therefrom.

As would be expected with the water supplies of Wisconsin taken from rivers and lakes, from springs and wells in the glacial drift, and from wells in the underlying sedimentary rocks, the total hardness varies over an extremely wide range. The softest water reported has a hardness of 28 p.p.m., and the maximum hardness of record is 1,210 p.p.m.

The range of hardness in each type of source is comparatively wide, as shown on Table 1. The surface supplies vary from 28 p.p.m. hardness for Wisconsin river water to 190 p.p.m. for Elkhart Lake water, and the spring supplies from 46 to 325 p.p.m. The dug well supplies range in hardness from 35 p.p.m. to 383 p.p.m. The widest range occurs in the drilled wells, from a minimum hardness of 28 p.p.m. to the extreme maximum of 1,210 p.p.m.

Figure 1 shows the distribution of hardness of the well supplies in Wisconsin. The softer well water is in the less populated northwest half of the state. The harder water occurs in the southeast half of the state, where the greater part of the inland cities are located.

A convenient classification of the hardness of water supplies, from the standpoint of the desirability of softening for household use, is as follows:

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A paper presented on October 15, 1940, at the Wisconsin Section Meeting, Manitowoc, Wis., by Louis R. Howson, Consulting Engineer, Alvord, Burdick and Howson, Chicago.

TABLE 1

*Selected Wisconsin Public Water Supplies Arranged to Show Range of Hardness of Each Type of Supply*

| SUPPLIES | | HARDNESS |
|---|---|---|
| SURFACE SUPPLIES | | |
| Town | Source | |
| | | *p.p.m.* |
| Portage | Wisconsin River | 28 |
| Ashland | Lake Superior | 44 |
| Neillsville | Black River | 52 |
| Hurley | Lake Lavina | 56 |
| Merrill | Prairie River | 82 |
| Florence | Fisher Lake | 108 |
| Sheboygan | Lake Michigan | 123 |
| — | Keshena Lake | 144 |
| Oshkosh | Lake Winnebago (and Fox River) | 160 |
| — | Elkhart Lake | 190 |
| SPRINGS | | |
| Town | | |
| Pence | | 46 |
| Medford | | 110 |
| Norwalk | | 218 |
| Delavan | | 325 |
| DUG WELLS | | |
| Town | Depth of Well | |
| | *ft.* | |
| Biron | 14 | 35 |
| Blair | 14 | 128 |
| Waupaca | 35 | 253 |
| Stratford | 35 | 383 |
| DRILLED WELLS | | |
| Town | Depth of Well | |
| | *ft.* | |
| Colfax | 233 | 28 |
| Barron | 400 | 102 |
| La Farge | 500 | 203 |
| Oconomowoc | 78 | 305 |
| Menomonee Falls | 1,100 | 400 |
| Manitowoc | 52 | 505* |
| Kewaunee | 187 | 620 |
| Plymouth | 510 | 735* |
| Neenah | 675 | 870† |
| Neenah | 666 | 1,030† |
| Sheboygan Falls | 421 | 1,210 |

* Well producing hardest water. † Abandoned in 1938.

| HARDNESS RANGE | SUITABILITY FOR DOMESTIC USE |
|---|---|
| Less than 60 p.p.m. | Soft water |
| 60 to 120 p.p.m. | Hardness not objectionable |
| 120 to 180 p.p.m. | Hardness objectionable |
| Over 180 p.p.m. | Hardness very objectionable |

For simplicity, and because of the fact that softening a water in the 60 to 120 hardness range for domestic use is not yet generally considered of sufficient importance to justify it, the 0 to 60 p.p.m. and 60 to 120 ranges are combined in a single range—0 to 120—in the following discussion.

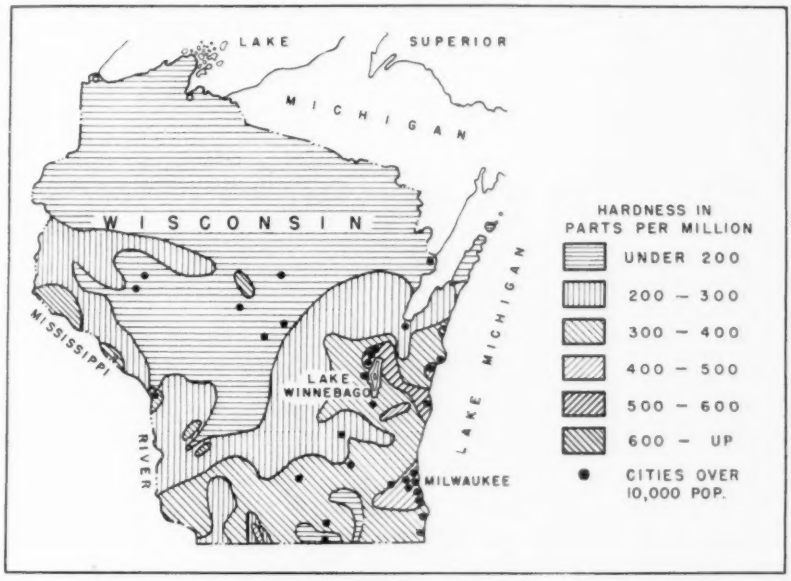


FIG. 1. Distribution of Hardness in Wisconsin Well Waters (after H. M. Olson)

## Distribution of Hardness According to Population Served

As of January 1, 1940, there were reported to be 282 Wisconsin towns and cities provided with a public water supply. These communities had a total of 1,749,000 population in 1930. In other words, approximately 60 per cent of the state's population is served by public water supplies. There were 502 incorporated places in Wisconsin

TABLE 2

*Distribution of 1930 Wisconsin Public Water Supply Population by Size of Town and Hardness of Water*

| SIZE OF TOWN | RANGE OF HARDNESS | | | | | | | | TOTAL | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0–120 p.p.m. | | Lake Michigan* | | 120–180 p.p.m. | | Over 180 p.p.m. | | | |
| | No. of Towns | Population | No. of Towns | Population | No. of Towns | Population | No. of Towns | Population | No. of Towns | Population |
| Less than 1,000 | 14 | 9,240 | 1 | 470 | 15 | 10,670 | 91 | 58,970 | 121 | 79,350 |
| 1,000 to 5,000 | 23 | 47,560 | 2 | 7,860 | 11 | 20,490 | 79 | 178,110 | 115 | 254,020 |
| 5,000 to 10,000 | 7 | 54,840 | 1 | 5,360 | 2 | 14,090 | 9 | 58,460 | 19 | 132,750 |
| Over 10,000 | 3† | 86,160 | 10 | 828,610 | 4 | 89,620 | 10‡ | 278,535 | 27§ | 1,282,925 |
| Total | 47 | 197,890 | 14 | 842,300 | 32 | 134,870 | 189 | 574,075 | 282 | 1,749,045 |
| Per cent of total water supply population | | 11.3 | | 48.2 | | 7.7 | | 32.8 | | 100 |

* Hardness 122–134 p.p.m.

† 7 in 1940, including 1 city with water softening.

‡ 12 in 1940.

§ 33 in 1940.

Note: 13 towns and cities with water softening are included above on the basis of hardness of the raw water supply.

in 1930 with a total population of 1,840,818. Assuming that all of the 282 communities noted above as having public water supplies are incorporated, it follows that 56 per cent of all incorporated Wisconsin places and 95 per cent of their total population are served by public water supplies. The 228 towns and villages without public water supply average less than 420 in population.

The above total population served by the public water supplies has been distributed as shown on Table 2, in order to show the total population using water of each of the above hardness classifications. The population is further broken down to show the total in communities of less than 1,000, from 1,000 to 5,000, from 5,000 to 10,000, and over 10,000. In this table, the figures are based on the 1930 census because the complete returns for the 1940 census are not yet available (October, 1940).

About 198,000 people, or roughly 11 per cent of the urban population, have the benefit of a water supply under 120 p.p.m. hardness. Communities making up this total would not generally derive very great benefit from water softening.

Because more than 842,000 people, 48 per cent of the Wisconsin urban total, use the Lake Michigan supply and because the hardness of Lake Michigan water is on the border line between the first and second hardness classifications, we have shown the population supplied from Lake Michigan separately. There is no present demand for softening the Lake Michigan water supplies, but the preponderant importance of this source in Wisconsin justifies careful consideration of the economic benefits that would accrue from softening. In the new Milwaukee plant the design was so developed as readily to include softening, if desired at a later date.

There remain the population groups served by supplies in the intermediate and high hardness ranges. In the former group, the total population served is about 135,000 or 7.7 per cent of the urban total, and in the latter group 574,000 or nearly 33 per cent of the Wisconsin urban population. The communities in these two groups would benefit very substantially by softening of the public water supply. These communities in total represent over 40 per cent of the urban population of Wisconsin, including the third largest city, most of other important interior cities of Wisconsin, and three-fourths of the cities and towns under 10,000 population.

In summary, it may be said that, including the Lake Michigan supplies, water softening would benefit to a greater or lesser extent over

TABLE 3

*Wisconsin Cities Over 10,000 Population With Soft Water Supply,* as of January 1, 1940*

*(Average Hardness Under 120 p.p.m.)*

| CITY | 1940 POPULATION | SUPPLY SOURCE | HARDNESS |
|---|---|---|---|
| Ashland | 11,304 | Lake Superior | 44 |
| Eau Claire | 30,517 | Drilled Wells | 39–63 |
| Superior | 34,944 | Infiltration Wells | 57 |
| Wisconsin Rapids | 11,396 | Inf. Tile and Dug Wells | 46–71 |
| Chippewa Falls | 10,240 | Springs | 64 |
| Neenah | 10,642 | Fox River, *softened* | 70 |
| Wausau | 26,748 | Dug, Drilled and Gravel Wall Wells | 88–136 |
| Total | 135,791 | | |

* Including one city with water softening.

TABLE 4

*Wisconsin Cities Supplied from Lake Michigan**

| CITY | 1940 POPULATION | | HARDNESS |
|---|---|---|---|
| Marinette | | 14,083 | 122 |
| Two Rivers | | 10,229 | 129 |
| Sheboygan | | 39,881 | 123 |
| Port Washington | | 3,693† | 126 |
| Fox Point | 474† | | |
| Whitefish Bay | 5,362† | | |
| Shorewood | 15,085 | | |
| West Milwaukee | 4,168† | | |
| West Allis | 36,146 | | |
| Milwaukee | 589,558 | | |
| Total supplied by Milwaukee | | 650,793 | 128 |
| South Milwaukee | | 11,115 | 134 |
| Cudahy | | 10,550 | 131 |
| Racine | | 67,159 | 130 |
| Kenosha | | 48,464 | 132 |
| Total | | 845,967 | |

* Including 4 under 10,000 population.

† 1930 Census.

88 per cent of the urban population of Wisconsin, or more than 1,550,000 people. Water softening of all the hard supplies, excluding Lake Michigan, would benefit a total urban population (1930 census) of about 708,000.

TABLE 5

*Wisconsin Cities over 10,000 Population Using Hard Water, as of January 1, 1940*

| CITY | 1940 POPULATION | SUPPLY SOURCE | HARDNESS |
|---|---|---|---|
| Hardness Range 120–180 p.p.m. | | | |
| Appleton | 28,398 | Fox River | 140 |
| Menasha | 10,473 | " " | 162–170 |
| Oshkosh | 39,054 | Lake Winnebago | 160–164 |
| Stevens Point | 15,751 | Dug & Dr. Wells | 120–130 |
| Total | 93,845 | | |
| Hardness over 180 p.p.m. | | | |
| Beaver Dam | 10,331 | Drilled Well | 284–296 |
| Beloit | 23,720 | " " | 280–316 |
| Fond du Lac | 27,156 | " " | 310–340 |
| Green Bay | 45,840 | " " | 215–245 |
| Janesville | 22,866 | Dug & Dr. Wells | 370 |
| La Crosse | 42,441 | Wells in Sand | 295–345 |
| Madison | 66,802 | Drilled Wells | 300–380 |
| Manitowoc | 24,299 | Gravel Wall Wells | 235–505* |
| Marshfield | 10,357 | " " " | 182–210† |
| Watertown | 11,270 | Drilled Wells | 295–330 |
| Waukesha | 19,205 | " " | 305–490 |
| Wauwatosa | 36,146 | " " | 395–590 |
| Total | 340,433 | | |

* Could use Lake Michigan.

† Except 1 of six wells, 144 p.p.m. hardness.

## Hardness in Principal City Water Supplies

Three-fourths of the Wisconsin urban population with public water supplies lives in the 33 cities of more than 10,000 population in 1940. The cities in this group are shown on Tables 3, 4 and 5, classified as to hardness of the water supply. Table 3 shows the 7 cities with a soft water supply, ranging from 39 to 88 p.p.m., the total population so served amounting to approximately 135,000.

At the border line between the soft water group of cities using water under 120 p.p.m. hardness and the group using moderately hard water ranging from 120 to 180 p.p.m., is the group of cities supplied from Lake Michigan. Including 3 under 10,000 population of the 5 communities supplied by the City of Milwaukee, and one other city under 10,000, a total of 14 communities with an aggregate population of about 846,000 take water from Lake Michigan. These cities are shown on Table 4.

The next group, with water from 120 to 180 p.p.m. hardness, exclusive of the Lake Michigan supplies and Neenah, which has water softening, includes 4 cities with a total population of approximately 94,000, as shown in the upper part of Table 5. Below this are shown the 12 cities with a total population of about 340,000 which have public water supplies ranging in hardness from 182 to a maximum of 505 p.p.m.

The cities over 10,000 population are summarized as follows:

| TABLE NO. | HARDNESS RANGE | NUMBER OF CITIES | 1940 TOTAL POPULATION |
|---|---|---|---|
| 3 | 0–120 | 7 | 135,791 |
| 4 | 122–134[1] | 10 | 845,967[2] |
| 5 | 120–180 | 4 | 93,845 |
| 5 | 180 and over | 12 | 340,433 |
| Total of hardness 120 and over (Table 5)..... | | | 434,278 |

[1] Lake Michigan supplies only.

[2] Includes the 1930 population of 4 communities under 10,000.

The cities shown on Table 5 (with a combined total 1940 population of about 434,000) would derive substantial benefits from water softening. The same is true of the approximately 207 cities and towns of less than 10,000 and having a combined population over 340,000 in 1930, as shown on Table 2. Thus a total population in Wisconsin of at least approximately 774,000 would benefit by water softening if applied to all Wisconsin communities having supplies harder than 120 p.p.m., but omitting Lake Michigan supplies.

Figure 2 shows graphically the 1930 population of Wisconsin served by public water supply of a given hardness or greater. The peculiar shape of the curve is due to the very large population supplied with Lake Michigan water in the narrow range of about 122 to 134 p.p.m. hardness.

This diagram shows that the 1930 population supplied with water of 135 p.p.m. or greater hardness exceeds about 760,000.

### Cost of Hard Water to Urban Population

People living in hard water regions are apt to take the disadvantages of hard water for granted and not realize the extent of the inconvenience and cost of using hard water. People who come to hard water localities from places where soft water is in general use (such as Boston and New York where the public supply has a hardness of 15 to 50 p.p.m.) have a better appreciation of the incon-

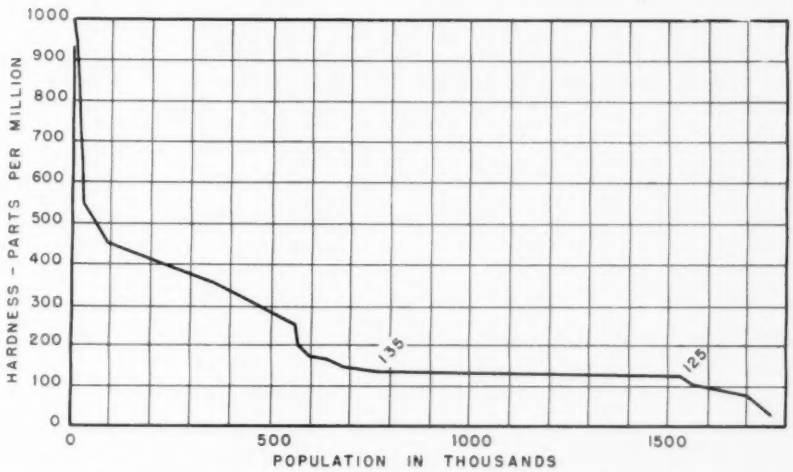


FIG. 2. Wisconsin Population (1930) Supplied by Public Water Supplies of a Given Hardness or Greater

venience of hard water. Even they, however, probably do not realize the aggregate cost of the wasted soap and of the many other indirect costs that accompany the use of hard water.

Among the many items of cost due to hard water are the following:

Increased soap consumption.
Reduced life of clothing and fabrics that are laundered.
Reduced life of heating coils, automobile radiators and other equipment using hot water.
Reduced efficiency of heating coils, etc.
Cost of cisterns and double plumbing.
Cost and inconvenience of household water softeners.

An attempt is made herein to express these items of hard water cost in dollars. It must be recognized that, being generally developed, they may not precisely fit any particular city. They are, however, believed indicative of conditions generally prevailing in Wisconsin. The estimated costs have been confined to those supplies having a hardness greater than 120 p.p.m., and omitting the Lake Michigan supplies. The estimate of soap cost assumes that each supply affected will be reduced to 85 p.p.m. total hardness. The *annual* cost of hard water to these 774,000 people now using it is estimated approximately as follows:

| | |
|---|---|
| Added soap cost | $1,200,000 |
| Increased cost of furnace heating coils | 150,000 |
| Increased cost of gas water heaters | 320,000 |
| Cost of maintaining cisterns and double plumbing | 400,000 |
| Cost of maintaining and operating household water softeners | 440,000 |
| Reduced life of clothing and laundered fabrics | 620,000 |
| Fuel loss, household water heaters | 50,000 |
| Increased cost of auto radiators | 500,000 |
| Total | $3,680,000 |

The above estimate of *soap consumption* is based upon a very careful survey made in the Illinois State Water Survey several years ago, comparing the per capita soap cost in several cities of about equal size but differing in hardness of the public water supply. This was reported by Hudson and Buswell in the JOURNAL.* These studies are the basis of the diagram, Fig. 3, from which the cost of soap saved by reducing the hardness of the water supply by any given amount can be estimated with considerable accuracy.

The other items are necessarily based on estimates from various sources in which opinion has unavoidably entered to a greater or lesser extent. It is believed, however, that the composite figures from widely scattered sources give a good idea of the magnitude of the losses due to these particular items.

The estimate with respect to the effect of hard water on auto radiator expense was secured from a questionnaire sent to dealers in one of the medium priced widely used cars. The cities in which these dealers operate were selected so as to give as wide a range in water hardness as practicable. It was found that the life of an auto

* Soap Consumption and Water Quality. Jour. A. W. W. A., **24**: 859 (1932). Quality of Water and Soap Consumption. Jour. A. W. W. A., **25**: 645 (1933).

radiator was about twice as long where the water supply had a hardness below 85 p.p.m. as it was where the hardness exceeded 300 p.p.m. The annual advantage of soft over hard water was approximately $3 per car. On the assumption that the 774,000 people using hard water owned automobiles in accordance with the national average of one car for each 4.4 people, the annual total excess cost of hard water would be over $500,000.

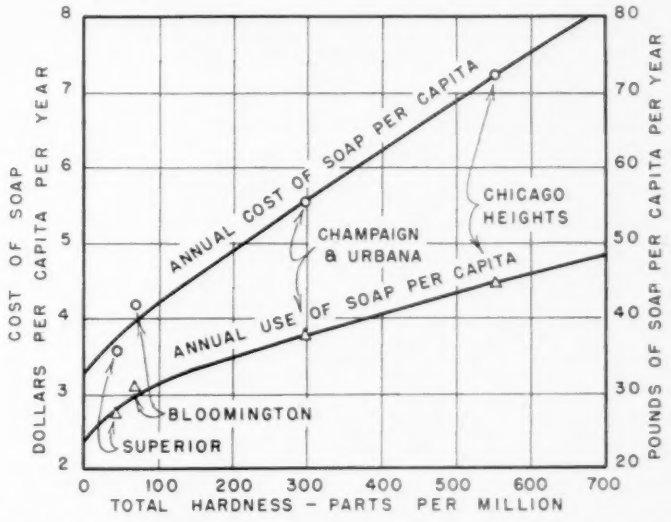


FIG. 3. Relation of Hardness of Water to Soap Cost in Superior, Wis., and Bloomington, Champaign, Urbana and Chicago Heights, Ill.

NOTE: Cost of soap was found to be 12.8¢ per lb. at Superior, with 45 p.p.m. hardness; 14.24¢ per lb. at Bloomington, with slightly harder water; 14.7¢ per lb. at Champaign and Urbana, with 298 p.p.m.; and 15.9¢ per lb. at Chicago Heights, with 555 p.p.m. This indicates that the harder the water the more expensive per pound for the soaps used to break down the hardness. It is believed therefore that the Buswell-Hudson use of the actual survey figures of soap purchases (Jour. A. W. W. A., **24:** 859 (1932)) is more reliable for use with waters of varying hardness than the uniform figure of 16¢ per lb. used by Olson (Jour. A. W. W. A., **31:** 607 (1939)).

Similarly, replies to questionnaires sent to plumbers in various cities disclosed that the life of furnace coils and gas heaters was but 20 to 30 per cent as long in a city where the hardness exceeded 300 p.p.m. as it was where the hardness was 85 p.p.m. or less.

These factual data were assembled, evaluated in dollars per unit and applied to the number of units per 1,000 population which data

or inquiry disclosed would probably represent average Wisconsin conditions. For illustration, replies from six Wisconsin cities showed 17.7 per cent of dwellings were stove heated, 61 per cent furnace heated and 57 per cent had hot water heaters of some type. Applying these ratios to the dwellings of the 774,000 people using hard water and using the unit prices secured from the questionnaires, the respective totals were derived.

The above total of $3,680,000 is not a complete figure. It does not include, for instance, the increased cost of plumbing maintenance, increased wear and tear on cooking utensils, all losses to industry due to hard water, etc.

An incidental benefit of water softening in the case of many of the Wisconsin well waters is that, if lime softening is used, the advantage of iron removal or of iron and manganese removal can be had with relatively minor additional cost. The objection to waters with a high iron content and with resulting tendency to stain laundry and fixtures makes the removal of iron in these instances an important advantage that goes along with the lime softening process.

### Present Status of Municipal Softening

A small beginning has been made in the softening of public water supplies in Wisconsin. The cities and towns reported as having water softening, as of January 1, 1940, are shown on Table 6. There are at present 13 communities ranging from 600 to a little over 10,000 population which have water softening, the total population so supplied being approximately 50,000. This is less than 3 per cent of the total population having public water supplies and less than $6\frac{1}{2}$ per cent of the population that would benefit very substantially by water softening. The towns which have resorted to softening include one supply with a reported raw water hardness from 104 to 134 p.p.m. In most cases the raw water hardness is in excess of 250 and ranges up to 375 p.p.m. Nine of the thirteen towns having municipal water softening include iron removal in the treatment.

All of these water softening plants are comparatively new, the first having been installed in 1933. It is a striking fact that most of the plants serve the smaller towns, the communities with water softening averaging under 4,000 population. It is thus evident that water softening of public supplies in Wisconsin is in its infancy.

Some of the larger cities are now considering softening. At Madison it has been a live question for years. La Crosse has recently had an investigation and report made. Appleton is building a new reservoir and chemical storage and feeding facilities to adapt its present filtration plant to softening.

### Water Softening by the Consumer

The great advantages of soft water have resulted, in the last few years, in a very rapid increase in the installation of residential water

TABLE 6

*Wisconsin Cities Using Softened Water, as of January 1, 1940*

| CITY | YEAR SOFTENING INSTALLED | 1930 POPULATION | SOURCE SUPPLY | HARDNESS | |
|---|---|---|---|---|---|
| | | | | Raw | Tap |
| Antigo | 1939 | 8,610 | Dug Wells | 104–134 | 75 |
| Columbus | 1933 | 2,514 | Drilled Wells | 272 | 38 |
| Elkhorn | 1939 | 2,340 | Drilled Wells | 268–311 | 80 |
| Evansville | 1933 | 2,269 | Drilled Well | 350 | 47 |
| Galesville | 1935 | 1,069 | Drilled Wells | 250 | — |
| Greendale | 1938 | 1,800 | Wells | 476 | 102 |
| Lomira | 1940 | 605 | Well | 380 | — |
| Neenah | 1938 | 10,642* | Fox River | 141–185 | 70 |
| Pardeeville | 1939 | 873 | Well | 310 | — |
| Platteville | 1938 | 4,047 | Wells | 315 | 80 |
| Waupun | 1935 | 5,768 | Drilled Wells | 373 | 37 |
| Williams Bay | 1938 | 800† | Gravel Wall Well | 355 | 95 |
| Lake Town | 1939 | 7,000 | Drilled Wells | 400–424 | 104 |
| Total | | 48,335 | | | |

* 1940 census.

† Winter population. Summer population approximately 5,000.

softeners of the zeolite type, which are regenerated with salt. The high cost of installing and operating these units will prevent a large portion of the consumers in the average community from enjoying the benefits of soft water, if it is left to the consumer to finance his own installation.

The City of Madison with 5,000 household softeners serving about one-fourth of the consumers probably has as large a proportion of private water softeners as any city in Wisconsin. Presumably, the ratio of water softeners to population might still increase substantially

at Madison. While it is too soon to estimate the saturation point in a development of this kind, it is on the other hand, probably a safe assumption that a considerable proportion of the population will always be without water softeners on their own premises.

Even now, with only one-fourth of the population served, it is estimated that approximately $375,000 has been invested in water softening equipment in Madison. This is exclusive of duplicate plumbing costs. It is estimated further that the annual cost of water softening as now practiced in Madison, serving but one-fourth of the population, amounts to approximately $90,000. This figure includes interest, depreciation, maintenance, and operation, with no allowance for any cost of attendance for operation.

### Comparative Costs of Municipal and Household Softening

If the entire Madison supply were softened before delivery to the consumers, it is estimated that the necessary investment in the water softening plant would total approximately $600,000. The annual cost of water softening, reducing the hardness from an average of approximately 340 p.p.m. to 85 p.p.m., is estimated at approximately $100,000 including all fixed charges. The water delivered to consumers would be satisfactory for domestic use and for practical purposes would be substantially comparable to the average water enjoyed now by those having zeolite softeners. With municipal softening, *all* of the consumers would enjoy the benefits of soft water at the same annual expenditure now made by the one-fourth with individual water softening, and the latter would save much of their present soft water expense and all its inconvenience.

Using Madison as an illustration, it is apparent that:

1. A municipal softening plant can be built for less than one-half the aggregate cost of installing private softeners to serve the entire community.

2. An entire community can have soft water through a municipal plant at an annual cost no greater than that necessary to serve one-fourth of the consumers through individual zeolite softeners.

3. The cost of softening water in a municipal plant would be approximately the same for the average consumer as the salt bill alone for a household softener. In addition, the household installation would require maintenance and replacement which over a period would make the cost at least three times as great as it would be if the consumer were served by a municipal plant.

4. It will cost the average consumer only two-thirds as much for municipally softened water as the salt and maintenance for an individual zeolite softener on which the interest and depreciation would also have to be met.

### Cost of Municipal Softening Plants

In order to give some general idea as to the cost of municipal softening plants in the central west under conditions prevailing as of 1940, a curve has been prepared (Fig. 4) based on the costs of a number of plants serving communities ranging in population from 2,500 to 62,000.

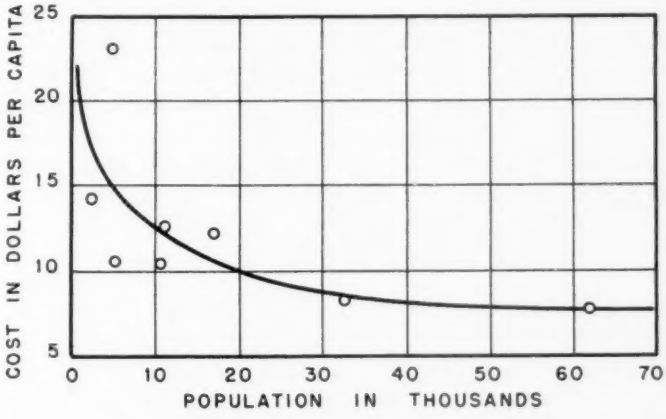


FIG. 4. Per Capita Cost of Softening Plants

Local conditions will, of course, affect the plant cost and in many individual instances the departure from this curve might be considerable. However, as a basis for estimating the cost of municipal water softening to Wisconsin communities generally, this curve is believed to be satisfactory. The costs shown are for the plant only, and are exclusive of land and costs such as bringing the raw water supply to a common point as may be necessary for a city supplied by widely scattered wells.

It will be noticed that in the larger cities, such as Madison, the cost per capita of the water softening plant proper may be below $8, whereas in a town one-third the size of Madison, the cost would be approximately $10, and for a town of 5,000 population it might be as high as $15 per capita or practically twice the cost for Madison.

The total plant investment necessary for softening all of the Wisconsin municipal supplies with a hardness greater than that of Lake Michigan water is estimated at approximately $9,590,000. This investment would serve a total population of approximately 774,000 at an average investment of $12.50. It is estimated that the aggregate cost of individual water softeners to serve this same population would be approximately $14,400,000.

It is estimated that the annual cost of state-wide municipal plant softening would be approximately $1,420,000. This includes interest at 3 per cent, depreciation at 3 per cent, chemicals at $10 per mil. gal. per 100 p.p.m. hardness removed, operating labor and maintenance.

This is equivalent to $1.83 per capita per year as a state-wide average for softening 335 p.p.m. water to 85 p.p.m. This figure will be higher for small cities and lower for large. La Crosse was estimated at $1.55 per capita. It will also be higher for harder waters and lower for softer, about half of the above cost being for chemicals.

The gross savings due to water softening may be approximated by multiplying the Hudson-Buswell soap savings estimate by three and the net savings by multiplying by two.

### Savings Effected by Municipal Softening

It has been estimated herein that the savings due to municipal softening of the harder water supplies in Wisconsin will amount to $3,680,000 per year. To accomplish this saving, it will be necessary to spend $9,590,000 for treatment plants on which the interest and depreciation, operation and maintenance will involve an annual expenditure of $1,420,000. The net saving possible to realize will, therefore, be $2,260,000 per year. Would any private industry hesitate to expend $9,590,000 in an enterprise which after paying interest and depreciation would yield a net profit of $2,260,000, or nearly 24 per cent annually?

The magnitude of this annual saving of $2,260,000 may be visualized as follows:

1. This saving would be sufficient to build 22 ten-room public schools each costing $100,000, every year.

2. This saving would be sufficient to build approximately 75 miles of 18-foot concrete road at $30,000 per mile each year.

3. This saving would be sufficient to purchase approximately 2,500 automobiles per year, each costing $900, or expressed another way,

every automobile owner in Manitowoc could have a new car every second year from the savings in state wide municipal plant softening.

4. This saving would effect net savings to each consumer of over $10 per year. In other words, his savings due to softening would very largely pay his present water bill.

5. This saving would be sufficient to give 2,200 students a sum of $1,000 each per year for higher education.

6. This saving would be sufficient to build a complete water works each year for a city of 30,000 to 40,000 population.

The real questions are not what will it cost to soften, or can a city afford to soften a hard water supply, but rather can a city with a hard water supply afford not to soften that supply in a municipal plant?

This paper could not have been written without the generous co-operation of Messrs. L. F. Warrick, Leon Smith, W. C. Staeffler, James W. Myers, Jr., W. A. Peirce, John Barth, H. M. Olson, a large number of automobile distributors, plumbers, and operators of gas utilities—all of whom furnished data and comments essential to the discussion. Their assistance is gratefully acknowledged.

**Discussion by *H. M. Olson.*** * Mr. Howson has given us a very interesting picture of what can be accomplished through municipally softened water supplies, but has used only part of the actual savings and benefits as stated and is very conservative in the statements made. One thing of great importance that should not be overlooked is the increase in consumers and increase in per capita consumption of water resulting from a softened water supply.

A great many of the zeolite plants, after four to seven years use, are found to be too small and are being enlarged by the simple method of adding additional units, notably at Lancaster, Ohio; Cuyahoga Falls, Ohio; Ambridge, Pa.; and New Bremen, Ohio. Elmore, Ohio, with no increase in population showed a per capita consumption increase in three years time of 16 per cent. Other communities record per capita consumption increases of from 8 per cent to as high as 35 per cent.

Municipal water softening plants in the United States as of January 1, 1940, totaled 536 in 534 cities, serving approximately 750 separate communities with softened, iron-free water to about 9,500,000 people.

* Promotion Engineer, Ohio Salt Co., Wadsworth, Ohio.

The approximate number of plants now (October 1, 1940) is 550; the drop-off in increase of plants is due possibly to lack of Government funds and the war scare. However, many new installations, independently financed, are now being made or are being considered. New applications of treating equipment, both lime and soda ash, and zeolite, have stimulated desire both from the installation and operating angle.

New developments in prospect in the near future include new methods for fluoride removal; a new type chemical precipitation system utilizing a 5- to 15-minute detention period (solids are plated out on grains of sand and the sludge dewatered and carted away for disposal); new types of resinous zeolites for calcium, magnesium, iron and manganese removal by sodium cycle as well as hydrogen acid cycle for removing total solids with high exchange capacity, (15,000 to 18,000 grains and a 0.3- pound salt/1,000 grains removal).

A survey is now being conducted which will give the answer to many questions on the operation of softening plants. The data are to be published soon in the JOURNAL.

# Public Water Supply in the Civil Defense Program

*By Harry E. Jordan*

ONE of the prime essentials in a well balanced civil defense program is adequate provision for the protection and, in case of disaster, the prompt restoration of the public water supply. This is true because of the dual uses of such water both for domestic needs and for fire fighting. The water department authorities, therefore, have a definite responsibility for planning proper plant and facilities to insure, for the areas under their jurisdiction, the minimum of interruption of water service that may result either from disaster or military action.

In this latter connection, much can be gained from European experience in the present war. The most conspicuous change in the method of destruction of property which has developed during the present European war results from the use of aircraft for bombing industrial and civilian housing areas. This transfers, to locations in which a large concentration of population exists, part of the destruction which in former times was confined to battle areas and to soldiers in uniform.

It appears that in the attacks on Belgium, France and Holland, where resistance of a counter air force was less effective than it has been in England, mass attacks on concentrated industrial and housing areas were carried on so that complete demolition of the structures was effected. A combination of high explosive and incendiary bombs, the action of which made possible the breakdown of structures and the rapid spread of fire in the attacked areas, was used.

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A memorandum prepared by Harry E. Jordan, Secretary, A.W.W.A., for the use of the U. S. Public Health Service as part of a publication on National Defense; revised by J. K. Hoskins, Senior Sanitary Engineer of the U. S. Public Health Service. The statements herein contained, insofar as they relate to policy, record the opinions of the author as an individual and are not to be taken as an official statement by the American Water Works Association.

A water department finds itself called upon, under these circumstances, to restore service promptly when street mains have been damaged, to cut off water wasting in demolished buildings, and to provide more than the normal quantity of water so that fire fighting can be carried on effectively.

## Water Departments—General

Improvements to, or extension of, the water supply system should be consummated with all speed in keeping with the ability of the department to carry on such work. It is becoming evident that the manufacturers of certain types of specialized equipment already have scheduled orders for material to be produced, which will employ their available facilities for some months to come. Unnecessary delay of orders related to contemplated improvements will undoubtedly subject them to further delay as a result of congestion of manufacturing facilities.

Water department facilities involving the use of power should be strengthened. Electric-power operated units should be provided with modern current input facilities. These facilities should be brought in from as widely divergent parts of the power system as is possible. This preparedness is necessary even though the electric utility service is interconnected with other electric power sources. Dependence upon a single non-interconnected power supply is dangerous The value of standby power equipment is recognized even under normal operating conditions of water departments.

Records of equipment replacements and consumable material used should be reviewed over a period of years. If definite policies concerning stock quantities are already in effect, they should be modified to increase the allowable reserve of stock items. Not only is the probability of their need greater under emergency conditions, but the ability to procure them on short order is already becoming definitely lessened. Priority listing, which may include water works supplies, may be put into effect and the commandeering of such supplies by Government authorities in time of emergencies is always possible.

The frequency of test operation of standby parts of operating equipment should be increased. By standby equipment is meant any operating unit not regularly used in day to day operations. The operating characteristics of valves which are normally open should be tested. Every possibility to operate without adverse results

should be known. The capacity of automatic devices to function according to plan should be a matter of record. If a schedule of test operations of such equipment is already established, it should be reviewed with the idea of increasing the frequency of such tests.

Stored material, such as coal, coagulants, etc., should be examined upon receipt and held in storage under such conditions that no foreign material of a destructive nature could be added to it.

## The Personnel

Most of our water works plants are exceedingly vulnerable to willful damage or sabotage. Consideration must therefore be given to visitors and trespassers as well as to employees.

Each employee of a water department should be investigated to the fullest possible extent so that his various affiliations and activities are recorded. Non-citizen employees require special handling. Steps which they may have taken toward the completion of American citizenship should be respected and encouraged. Lack of progress toward naturalization should be grounds for suspicion and possible dismissal. An employee whose attitude is known to be non-cooperative or disloyal to American interests should be detached from from the service. Promotion of a proper attitude among the employee group requires equivalent qualities among those whose duties require leadership in water department operation.

The presence of persons in and about water department properties should either be under positive control or be forbidden. The requirement that all persons visiting an operating water department plant should have a pass obtained from the central office is the simplest method of handling the visitor problem. The limitation of such passes to one-day use has already been established in certain water departments. In other departments, the addition of an identifying photograph to the pass is required.

Employees operating a unit of a water works property must be trained to note the actions of any person about the property who is not an employee. Strangers should be contacted immediately and engaged in conversation. Full information concerning their interest about the property should be obtained and if the slightest doubt develops, the individual should be taken into custody.

Employees should be encouraged to observe and report upon defects or weak points in plant operating equipment or upon deviations from normal operating conditions. It should be noted that the

common course of an experienced employee is to consider deviations from normal operating routine unimportant so long as the service of a particular piece of equipment is not completely interrupted. It is important under present circumstances to require the opposite attitude, since interruptions to operating equipment are unusually undesirable, and since the procurement of replacement parts is likely to become less easy than is normally the case. This is a definite problem of department management and requires considerable attention to make its values fully evident.

The need for, and use of, guards about water department properties should be reviewed. If guards are employed, their past employment record should be definitely recorded and absolutely satisfactory to the executives of the water department. Guards, if employed, should be a part of the water department personnel and subject to the unqualified control of the water department executives.

## Water Supply Structures

If the water department operates an impounding reservoir in connection with which there are control works and a dam, or if its supply is taken from a lake or large stream, an agreement should be established with sportsmen's organizations to the end that the members coöperate with the water department in the protection of the water supply. This is the very least that can be done under present conditions. The exclusion of licensed sportsmen from watershed areas may become necessary in certain parts of the country. All non-employees should be excluded from areas adjacent to dams, intakes, or treatment plants, unless they have been provided with an identifying pass and are in the area on approved business. Camping, picnicking, the parking of cars and similar recreational activities should be forbidden in the areas adjacent to dams, intakes and exposed aqueducts. The presence of persons engaged in such activities at any place about the water storage area should be under definite control.

Protective lighting equipment should be installed at dams, intakes, head houses and near exposed aqueducts. Such lighting equipment should be installed in a fashion that will promote the protection of the operating units rather than simply illuminating them for the information of improperly disposed persons. In such lighting systems, provision should be made for standby independent electric current generators.

Emergency control works and intake facilities about impounding reservoirs and dams should be planned so that, if an interruption to the regularly installed equipment occurs, restoration of service can be made promptly.

Men on duty about impounding reservoirs and dams should report promptly, to the department head, the identification markings on airplanes flying about or over any dams, intakes, or reservoir structures, for prompt transmittal to the local or state police so that an accounting for the activities of the aircraft may be obtained. It is imperative that this procedure be made routine, that the accounting be followed through to the satisfaction of the department executive, and that those in charge of airplane equipment develop a respect for the nature of water supply structures.

Isolated manholes, aqueducts and similar structures should be covered with earth whenever possible, so that the identification or location of the structure by unauthorized persons is difficult. Since the engineering division of any water department should be competent to develop a record of the precise location of any such buried equipment, there is no reason to continue the exposure to depredation of any water works property which can be concealed.

Radio communication systems or personal messenger service should be set up to supplement existing wire communication systems. If radio communication systems are installed they should be provided with independent power facilities. This is based upon the possibility that any interruption to existing wire communication systems would also involve an interruption to central station electric power sources. Thus, the emergency value of the radio communication systems would be destroyed. If the city or state police are equipped with radio communication systems, plans for coöperative use of such equipment should be developed.

### Water Treatment Plants

Coagulating basins, storage reservoirs and storage yards should be fenced. The general principle of preventing damage by persons on water works property requires that possible lines of easy travel in and about the plant be interrupted by adequate fences, walls or similar structures.

Possible increase of procurement difficulties should be guarded against by raising the standard stores allotment for reserves of coagulants, chlorine, and other supplies. A normal good manage-

ment policy establishes a stores reserve of about 150 per cent of the amount of material which could be brought to the operating unit within the time required for shipment from the producer to the plant. Under present conditions this reserve should be increased to from 200 per cent to 250 per cent of such an amount and such quantities should be considered in the light of the ability of the producer to maintain deliveries of material.

All coagulant feeding and chlorination equipment must definitely be known to be in operating condition. Actual tests of the performance ability of any reserve units are definitely required.

At least one chlorinator, generally of small capacity, should be mounted on skids or wheels so that under emergency conditions it can be moved quickly to any point in the water distribution system and used to treat a supply being provided under emergency and temporary conditions. This unit should not be held in unused reserve but should be functioning regularly as a part of the plant operating equipment until needed for emergency.

The frequency of water sampling from all sources and on all parts of the system should be increased. It is important that simple chemical tests (such as chlorides, alkalinity, etc.) be established so that the presence of water foreign to the regular supply can be determined promptly. The number of samples of water taken from the distribution mains and submitted to bacteriological examination should be increased. If storage reservoirs of finished water float on any part of the system, sampling of water from these sources should be as regular as that from any other part of the water supply.

The possibility of the absorption of poisonous war gases, such as mustard gas, by exposed bodies of water used for the public supply should not be overlooked.

## Water Distribution System

Reinforcements to the distribution system, which have been recommended by proper authorities and which are desired for the purpose of providing good service in all parts of the system, should be completed without delay. The ability of water plants to serve under emergency conditions definitely depends upon the addition of reinforcement structures, the need for which has been recognized by those who have studied the ability of the system to serve its customers.

Special attention should be given to the reinforcement of the

system in industrial areas. If the industries are engaged in the production of defense equipment, the need for reinforcement becomes imperative. Such improvements include not only adequate supply mains but a sufficient number of valves to control the piping system in small areas and a sufficient number of hydrants for the service required under the most severe conflagration conditions.

Tests of distribution valves and hydrants should be a matter of frequent routine and definite record. This again is an imperative part of the operation of a water department under stress conditions. Overseas war-time activities have demonstrated, beyond doubt, the need for a large number of control valves on water mains and for many hydrants connected to these mains.

Improvements in a water supply system, recommended by fire insurance engineers, should under no normal circumstances be deferred. If they are deferred, water department executives should have recorded by definite memoranda, the reasons for such deferment. Since it is apparent that one of the major problems associated with present day war activities is an increased number of fires, the ability of a water department to furnish water for fire fighting should not be under question by any competent authority.

Emergency maintenance crews and other proper authorities should be provided with distribution maps of the entire water system. The complete plan for the production of such records has been developed by a committee of the American Water Works Association (*Journal of the A.W.W.A.*, February 1940). The maps of the water department structures, naturally, should be kept under lock in a safe storage place. They should, however, be easily accessible to an authorized person whenever needed.

The personnel of emergency maintenance crews and the equipment provided for them should be increased. These crews should be equipped with special communication facilities, either of the radio or special messenger type, so that the availability of the crews to the supervisor of the activity is prompt. Under increasing emergency conditions the executives in the larger cities will be faced with the problem of providing dormitories and kitchen equipment for such crews. This practice is clearly recorded in England at the present time and plans should be made for its duplication in this country.

An ample supply of emergency repair materials and equipment must be available so that prompt restoration of service by distribution mains can be effected. This should include sections of tempo-

rary pipe and sleeve fittings, to be laid, if necessary, above ground. Special or close fitting of emergency pipe lines should not be necessary. The use of sleeve connections, ells and similar devices should make it possible for service to be restored to lines not more than eight inches in diameter within an hour after the crew reaches the scene of the break.

Interest in the health of the users of water from public service mains demands that, even under emergency conditions, protection against pollution be given parts of the water distribution system which have been taken out of service for repair. Addition of disinfecting material (such as liquid chlorine, HTH or Perchloron) to the water entering the repaired main, and its withdrawal through the water discharge of a connected fire hydrant, is not a difficult procedure. Repair crews must definitely be directed to conduct their operations in such manner as will protect the health of the water users.

Adequate portable lighting and pumping equipment, independently powered, should be available in sufficient quantities to make possible the availability of the equipment to every crew which might be assigned to emergency repair work.

Booster pumping stations, elevated reservoirs, standpipes and tanks require special handling and guarding to the degree that they are isolated from built-up areas or other parts of the water department's system. If any such structure serves a wide area and a considerable number of people, special guards should be employed to protect it.

If the mains of one water department or system lay adjacent to the mains of another department or system, a permanent connection should be made between the two properties. This connection can be valved off definitely, and meters installed, if necessary. The provision for emergency supply of water from one system to the other must be made and the authorization for the completion of such work should be obtained from any appropriate controlling body without delay.

## Emergency Supplies

Automotive equipment and utensils should be provided for the supply of a minimum quantity of water for persons in areas where water supply has been cut off. The personnel to operate the is equipment can in time of emergency be obtained from the commercial

department and the water treatment division of the water works property. The men should be trained for this special type of service and the people in the community should likewise be prepared to cooperate in any such service when it becomes necessary.

All available sources of water supply in private plants in any municipality should be listed. Their power facilities should be a matter of record; their availability under emergency conditions clearly demonstrated; and arrangements made for emergency service, if necessary. It is recognized that these provisions represent a deviation from the underlying policies of health authorities who do not now view with approval the cross-connection of private supplies to public mains. In an emergency, when the normal water supply may be curtailed or totally unavailable, normal considerations must give way to human needs. This does not, in any manner, imply that the cross-connection of private supplies to public mains is advocated as a normal engineering procedure.

### Emergency Disinfection of Drinking Water

Interruptions to normal distribution of drinking water may be caused by a number of circumstances during war times. Water which has been actively boiled will be safe to drink after it has been cooled. It should of course be stored in thoroughly clean containers.

Since gas and electricity supplies may as easily be interrupted as the public water supply, provision is needed for emergency sterilization of drinking water by use of chemicals. Materials which contain chlorine available for disinfection purposes are obtainable. Since the water, which may be at hand, may have been subjected to unusual pollution, it is advisable to add more of the treatment material than would be needed for well water or clear water from a surface stream.

High-test hypochlorite (sold under the trade name of HTH or Perchloron, etc.) may be purchased through many sales outlets which stock steam-laundry supplies. In great emergency, any modern laundry may become a source of a necessary amount for immediate use. Safety under emergency conditions would indicate the propriety of using one ounce in 5,000 gallons of water. If a distinct odor of chlorine does not persist for thirty minutes, a still heavier dose should be used. The chemical should be mixed thoroughly in the water being treated, and the mixture allowed to stand at least thirty minutes. As the water is being drawn off for use, hyposulfite

of soda may be added, in weight equal to the weight of the hypochlorite used for disinfection, to destroy the pungent chlorine odor and unpleasant chlorinous tastes.

The use of untreated water under emergency conditions is not the mark of a good citizen or a good soldier and may lead to the unnecessary illness of many persons at a time when preservation of health is of prime importance.

## Appendix

### General Organization to Meet Emergency Conditions

Many of the procedures outlined herein as steps to be taken in preparation for emergency conditions are, in fact, elements of the best operating practices of a utility in normal times. They should not be considered exclusively emergency measures, but to a large degree applicable to the regular training of public utility personnel so that they can meet the routine as well as the unusual demands upon a property.

While there may be a tendency to view the instructions as a manual of operation in times of war, they are also, in the main, a manual of operation in time of any disaster which interrupts, in any way, urban life and public service of a utility nature. Either earthquakes, floods, strikes, tornadoes, sabotage, or acts of war may set up the circumstances against the effects of which special preparedness measures are necessary.

This broad concept of the need for emergency planning justifies clearly the organization of a civil defense or emergency service committee in many cities and states within the United States.* Whether these groups be organized first within state lines or within municipal lines is unimportant. They should include key representatives from every public administrative department that has to do with police, fire, law, highway or street, health and similar phases of public administrative activities. They should also include key men representing each of the various utility services such as light, water, transportation, gas, and the like, rendered in any region. At the head of

* A list of state defense councils in action in December, 1940, appears on page 4 of the News of the Field Section of this issue of the Journal.

each organization should be placed an individual whose leadership qualities are evident to all concerned. Legislation should be enacted, either as state law or city ordinances, to make the work of the committee effective. The leader of the group should be titled "Civil Defense Commissioner," "Disaster Committee Chairman," or any other title that will develop public recognition of the nature of the service rendered by the committee and administered under his leadership.

A county or city group should formulate a general plan of procedure to cover the emergency handling of any or all community service which may, in part or whole, be interrupted by earthquake, flood, strike, tornado, sabotage or war. When an organization is established in any city or county, its form and nature should be reported to the state defense council. Each state, in turn, should, in due time, establish a defense council to coordinate all activities within the state so as to promote, to the most effective degree, the activities within the field of interest of these groups. The state defense councils should, in turn, report their form of organization and activities to the appropriate Federal agency.

Perhaps the best plan of city and county organization in the United States is the one adopted by Los Angeles City and County and formalized in an ordinance by the Council of the City of Los Angeles in November, 1933. This "Major Disaster Emergency Council" has established a complete organization and plan for action to deal with events which might develop to disrupt the ordinary organized public service in the Los Angeles County area. The complete text of Ordinance #73309 follows.

### Ordinance No. 73,309

An ordinance creating a Major Disaster Emergency Council and prescribing its powers and duties.

The People of the City of Los Angeles do ordain as follows:

SEC. 1. For the purpose of providing a co-ordinating organization and to be prepared to meet the needs of the citizens of Los Angeles, in the event some major physical disaster or catastrophe overtakes us, this ordinance is enacted.

SEC. 2. There is hereby created a Major Disaster Emergency Council which shall consist of the Mayor of the City of Los Angeles, a representative of the Los Angeles Chamber of Commerce, a representative of the Los Angeles Chapter, American Red Cross and all

the officers and members of each of the following enumerated units and their subcommittees, which shall perform the duties and functions imposed upon them by the provisions of Sec. 3 hereof, to wit:

(a) Co-ordination and Intelligence;
(b) Transportation;
(c) Communication;
(d) Personnel—American Legion;
(e) Law and Order;
(f) Fire;
(g) Water and Water Supply;
(h) Street Facilities;
(i) Medical, Health and Sanitation;
(j) Necessities of Life;
(k) Shelter;
(l) Rescue;
(m) Rehabilitation;
(n) Finance.

SEC. 3. It shall be the duty of said Emergency Council to undertake to co-ordinate all resources of the City of Los Angeles and the various citizens of said City, together with the resources of various corporations, business houses and associations doing business in the City of Los Angeles and by means of mutual co-operating and effort create a plan which would permit an effective and efficient method of utilizing all available resources and materials for relief of and the general welfare of the people of the City of Los Angeles in the event some major catastrophe, disaster, calamity, or cataclysm overtakes us and in formulating such plan said Council shall act in accordance with all laws to which the City of Los Angeles is subject and each and all members of said Council shall diligently undertake to acquaint himself with and become familiar with the responsibilities imposed upon him in the event of such emergency.

SEC. 4. The Mayor of the City of Los Angeles shall be Chairman of said Emergency Council and should he be absent from the City of Los Angeles or disqualified or unable to act for any reason, the powers and duties imposed upon the Mayor by the provisions hereof shall then be vested in the President of the City Council. The Mayor shall have the power, in his discretion, to declare when an emergency disaster exists and it shall be the duty of the City Council of the City of Los Angeles to conform and ratify the acts of the Mayor,

acting in his capacity as Chairman of the Emergency Council, during the condition of emergency.

The Mayor shall have the power, in the event of such emergency, to obligate the City of Los Angeles for the payment for any and all supplies, equipment, materials, food or other necessities of life that may be necessary for the purpose of alleviating suffering to the citizens of this City and for the purpose of protecting the citizens from the hardships anticipated in case such condition arises.

SEC. 5. Those persons acting as Chairmen of the Units enumerated in Sec. 2 hereof, after letters (a) to (n) inclusive, the Mayor, a representative of the Chamber of Commerce, and a representative of the American Red Cross, shall be known as "Emergency Disaster Executive Committee" and upon the declaration of the Mayor that an emergency exists, he shall forthwith establish communication between himself and the Executive Committee of said Council. The members of said Executive Committee shall be in charge of and direct the following divisions or units of the plan contemplated.

(a) *Intelligence division:* The Intelligence Division shall have charge of, and direct:

(1) The study of the organization.

(2) Co-ordination of all divisions and units or groups referred to in this ordinance.

(3) From time to time to make tests to determine the effectiveness of the plan created.

(4) To have charge of the publicity and dissemination of information to the public concerning the proposed functions of said Council, in the event of a major disaster and during the time of such disaster, if any may exist, to gather such information as may be necessary to permit the effective functioning of the plan contemplated hereby.

(5) The Chairman of the Intelligence Division shall be known as the Director of Coordination. It shall be his duty to maintain a permanent headquarters of this Major Disaster Emergency Council. He shall devote all his time to the emergency council. In addition to paragraphs 1, 2, 3 and 4 of Section 5-a, he shall keep in constant touch with the members, keep reports up to date, drill and instruct the various units, keeping them in touch with all the activities of the Emergency Council. The Chairman of the Intelligence Division shall receive such compensation for his services as such shall be fixed by the City Council.

(b) *Transportation:* The Transportation Division shall direct and have charge of the operation of all facilities of transportation including railroads, street cars, private automobiles, motor buses, taxicabs, airplanes and pilots, trucks and steamships.

(c) *Communication:* The Communication Division shall arrange for and direct the operation of all systems of communication which may be necessary or useful to insure the efficient functioning of the plan contemplated hereby.

(d) *Personnel—American Legion:* The Personnel Division shall have charge of and direct:

(1) The mobilization and stationing of all groups of ex-service men.

(2) Distribution of such personnel as may be available to other divisions for assisting such other divisions.

(e) *Law and Order:* Law and Order Division shall be under and subject to the control of the Los Angeles Police Department and shall have charge of and direct the enforcement of law and order, and shall assign the stations and direct the duties of the following:

(1) Department of Police, City of Los Angeles.

(2) Sheriff of Los Angeles County.

(3) Such members of the United States Army, Navy, Marine Corps, National Guard, Reserve Officers, R. O. T. C. and others as may be available.

(4) The Chairman of the Law and Order Division shall organize for the assistance and cooperation of the Police Department all outside groups as set forth in paragraph 3 of Section 5-e.

(f) *Fire:* The Fire Division shall be under and subject to the control of the Los Angeles Fire Department and said Department shall have the authority to do any and all acts vested in said department under provisions of Article X of the Charter of the City of Los Angeles, and in addition thereto, in case of necessity:

(1) The use and control of all explosives.

(2) The demolition of buildings or structures.

(3) The use and control of all oils and chemicals.

(4) Outside auxiliary fire fighting equipment.

(5) Direction of Public Utilities, such as Gas, Electricity, Ammonia, etc.

(g) *Water:* The Water Division shall have charge of and direct the maintenance of water supply, repairs to water system, control of reservoirs, and shall perform such duties as may be necessary to insure an adequate, uncontaminated supply of water.

(h) *Streets:* The Street Division shall have charge of and direct cleaning of debris from the streets and the keeping of such streets open for the purpose of permitting transportation thereover and the maintenance of sewer facilities.

(i) *Medical, Health and Sanitation:* The Medical, Health and Sanitation Division shall be under and subject to the control of the Los Angeles Health Department, and shall have charge of and direct:

(1) Hospitalization.

(2) Vaccination.

(3) Quarantine.

(4) Food Inspection.

(5) Sanitation.

(6) All physicians, surgeons and nurses.

(7) Medical and surgical supplies.

(8) The Chairman of the Medical, Health and Sanitation Division shall organize for the assistance and cooperation of the Health Department all outside physicians and surgeons, nurses and hospitals.

(j) *American Red Cross:* The American Red Cross shall have charge of and direct all American Red Cross activities as hereinafter set forth in Paragraph (k) to (o) inclusive.

(k) *Necessities of Life:* The Necessities of Life Division shall have charge of and direct, allocate and distribute all goods and wares commonly known and referred to as necessities of life, including food supplies, clothing, bedding, cooking equipment and service equipment.

(l) *Shelter:* The Shelter Division shall provide for shelter housing in temporary buildings or otherwise, small tents and other materials available for use for housing purposes, fencing, fuel supplies, sanitation, drainage, heating equipment such as stoves, etc.

(m) *Rescue:* The Rescue Division shall include all first aid, searching for disabled persons and dead bodies, transportation of sick or injured to hospitals, life guards.

(n) *Rehabilitation:* The Rehabilitation Division shall have charge of rehabilitation as provided for by the American Red Cross manual.

(o) *Finance:* The Finance Division shall have charge of and direct the collection of and distribution of all money contributions made available for use in furtherance of the plan contemplated hereby.

Nothing herein contained shall be construed as limiting or modifying in any way:

(1) The disaster relief responsibility of the American National Red Cross as imposed by the charter given to it by the Congress of the United States in 1905.

(2) The policies and procedures of the American National Red Cross in disaster relief as they have been, or shall be promulgated by the Central Committee or responsible officers of that organization.

SEC. 6. Each and all divisions hereinabove referred to shall be under the control and direction of a Chairman, and in the absence of express declaration to the contrary by a majority of the members of of the Executive Committee, and he shall do any and all acts imposed upon the Division of which he is in charge. The Chairman and two alternate Chairmen of each such Division, shall be appointed by the Mayor of the City of Los Angeles, subject to the approval of the City Council, and shall hold office for life during good behavior, provided, however, that should he resign or become disqualified to act, the vacancy created in such chairmanship or alternate chairmanship, shall be filled by vote of the majority of the members of the Executive Committee.

SEC. 7. In the event of the declaration of any emergency, it shall be the duty of the Mayor to notify all members of the Emergency Council of said declaration and all of such members shall forthwith assemble at Westlake Park.

The Emergency Council may make such rules and regulations as they may deem necessary for the purpose of putting the plan contemplated hereby into full force and effect, provided, however, that no person shall be paid any compensation for any service rendered hereunder, except as expressly provided for in this Ordinance.

SEC. 8. The Emergency Council shall authorize a badge or pass card as a means of identification, and said badge or card will be recognized by all police, sheriffs, and state officers, and others, as authority to pass in devasted area. Said badge or pass card will be numbered and signed for by members receiving same and will be issued only to regular members of the Emergency Council and only for the duration of their active service. The member receiving badge or pass card shall deposit with the Emergency Council an amount of money covering the cost of said badge or pass card, said money to be refunded to the member on return of badge or pass card.

SEC. 9. It shall be unlawful for any person other than a member of said Emergency Council to wear or carry about his person any such badge or pass card referred to in Section 8 hereof, or to wear or carry

about his person any badge or pass card of a design or pattern that may be taken for or confounded with the badge or pass card heretofore or hereinafter so adopted by said Emergency Council. Any person violating the provisions of this section shall be deemed guilty of a misdemeanor and upon conviction shall be punished by a fine of not more than $500.00 or by imprisonment in the City Jail for a period of not more than six months or by both such fine and imprisonment.

SEC. 10. The City Clerk shall certify to the passage of this ordinance by a unanimous vote and cause the same to be published once in *The Los Angeles Daily Journal.*

I hereby certify that the foregoing ordinance was passed by the Council of the City of Los Angeles by the unanimous vote of all members of said Council present, there being not less than twelve members present, at its meeting of November 22, 1933.

ROBT. DOMINGUEZ, *City Clerk.*

Approved this 4th day of December, 1933.

FRANK L. SHAW, *Mayor.*

*For abstracts of some recent articles on defense see page 171 ff.*

# Control of Water Well Drilling

*By Lewis S. Finch*

CONTROL of water well drilling is a broad subject, which must cover wells for municipal supplies; semi-public supplies, serving office buildings, parks, tourist camps, and the like; and private supplies, serving individual homes. Over 800,000 people in Indiana are served by public water supplies drawn from wells, and at least 20,000 of these people drink water of questionable quality, in spite of the fact that the State Health Department has been exercising control over public water supplies for many years. The health department exercises a degree of control over semi-public water supplies; but, at the present time, no attempt, except through educational channels, is made to control the development and maintenance of private wells.

For years it has been the practice of health departments throughout the country to exercise control over the installation of plumbing in private homes and public buildings even though, from the health standpoint, it is rather difficult to point out definitely the direct connection between health and most of the features covered by the various plumbing codes. Nevertheless, it is considered reasonable by the public in general that there should be plumbing codes and that they should be enforced.

In Indiana, one and one-half times as many deaths from typhoid fever occur in rural areas as in built-up areas where public water supplies, subject to control, are available. It can not be said, of course, that this preponderance of rural typhoid fever is altogether due to unsafe drinking water, but, undoubtedly, the rural water supply is a material factor.

In cities, the safety of public water supplies is carefully supervised, yet many of the city dwellers spend their working hours in office

A paper presented on April 4, 1940, at the Indiana Section Meeting, Lafayette, Ind., by Lewis S. Finch, Consulting Engineer, Indianapolis, Ind.

buildings and other structures where the available drinking water is neither subject to laboratory supervision nor drawn from wells located or constructed in accordance with accepted standards. It is true that the water is carried from the wells to the drinking fountains by water pipes installed according to code, and then is carried away from the fountains, and from the toilets by coded drains, but the well, itself, from which the water is obtained is usually outside the code.

A careful review of this situation indicates the necessity for more definite control over the methods used in locating, drilling and equipping such water supply wells.

Obviously, there are important factors in well construction which do not materially affect or even jeopardize public health, but which are important from the standpoint of public interest. Standard specifications covering well construction were prepared several years ago by the American Association of Water Well Drillers. These specifications have been quite helpful to engineers and others preparing well specifications, but their scope was not sufficiently broad to cover all features of well construction.

As has been pointed out by Harding in his paper on "Specifications for the Construction of Deep Wells" presented before the A. W. W. A. at its Atlantic City convention (Jour. A. W. W. A. **32:** 65 (1940)), the State Health Department of Wisconsin has developed and is enforcing a code which governs the drilling of water wells. This code establishes minimum construction standards and provides for the licensing of water well drillers. The regulations are excellent and if enforced should materially improve the quality of well water supplies in Wisconsin. Even in this case, however, there are very important angles involved in water well drilling which are not covered by the code because they do not, even indirectly, affect the public health.

A committee of the A. W. W. A. is at present preparing standard specifications for well construction. These specifications, when completed, will, undoubtedly, cover all phases of well construction and presumably could serve as the basis for a well drilling code which could be applied in a manner somewhat similar to that in which state plumbing codes are administered. Such a code could best be administered by the engineering divisions of the various state health departments, acting in co-operation with the state geology or natural resource boards. A new board or commission for controlling water

well drilling could, of course, be established by legislative enactment, but should that be done there would be a conflict of authority with the state health departments and, in addition, administrative expense would be materially higher.

Obviously it would be impossible and also undesirable for the state health departments to maintain direct supervision over the drilling of individual private wells, but it should be possible for them to give at least general supervision to well drilling methods and, where local health units are available, to provide a somewhat more direct degree of supervision.

### Considerations in Preparing a Code

If a program of control over water well drilling is to be initiated, it will be necessary to educate the officials and others involved in its operation to the fact that there is more to well drilling than merely punching or digging a hole in the ground. For example, most people, and even some water works men, are of the opinion that water is either available or not available at any given point in the ground; and, if the water is there, that the only operation which a driller must carry out to develop the supply is the drilling of a hole deep enough to find the water. This is far from the truth. No matter how skilled the driller may be, he cannot, of course, get water where there is none, but, if he is not sufficiently experienced and conscientious, he may not get water even though it actually is available and awaiting proper development. For example, there are many localities in Indiana where water cannot be obtained from rock wells, but where properly constructed and developed gravel or sand wells will yield an abundant supply. Drillers have been known to drill through and case off water-bearing gravel and sand formations and proceed to drill a dry hole in the rock. Obviously no code which will prevent such an occurrence can be drawn up, but, if the general standard of experience and ethics of the water well drillers is raised to a higher plane, such errors are not so likely to happen.

In the case of gravel and sand wells, it is necessary that the wells be carefully developed after drilling is completed, so that they will yield the maximum amount of water available in the formation. This may be accomplished in a number of ways—effectiveness of operation depending largely upon the experience, equipment and willingness of the driller. While this is not a matter which can be covered completely by a code, the equipment required and general

methods which should be adopted can be set forth. The balance of the problem must be met by raising the standards of performance of the drillers, either through education or through the encouragement of the conscientious driller who will employ all available methods of development.

Another popular fallacy held by many people is that the proper capacity of a well can be determined by pumping it a few minutes, or a few hours, if you will, without regard to drawdown; and by assuming, if the well continues to yield water at the end of the pumping period, that it will continue to supply this volume of water indefinitely.

Several years ago, the town of Milan, Ind., drilled wells and went through the motion of testing them. The system was completed upon the basis of the purported test and, after the system went into operation, the wells were pumped dry after a few days. No additional money was available for further well development, so the supply was abandoned and the practice of settling pond water and pumping it to the mains was resumed. Milan, today, has ten-year-old well pumps which have been operated for less than a month.

## Benefits of Control

Was the Milan fiasco the fault of the driller, and would a well drilling code have prevented this failure? In this instance, the actual well driller could not be blamed, but the well drilling contractor and the engineer were responsible for not having tested the well properly for one reason or another. Had a comprehensive code, which contained definite provisions with respect to capacity tests, been in effect, the failure to test these wells properly would have constituted a breach of contract and the town would have been in a better position, so far as recovering the money paid for the drilling is concerned. Had it been required that well drillers be licensed, the drilling contractor, probably, would have been more "conscientious" in reporting results of the pumpage tests to prevent having his license revoked.

Few Indiana drillers have adequate equipment for carrying out proper pumpage tests. Under a properly written code, requiring the licensing of drillers, the possession of adequate testing equipment should be requisite to the issuance of a driller's license. It should not be inferred from this statement that a "farm well" should be subjected to as extensive a test as that required for a public water

supply well, but a test of all new wells, at least sufficient to inform the owner of the capacity of the pump which properly should be installed, should be made and reported completely to the owner.

At present, in many cities where well water can readily be obtained, it is the practice of the owners of office buildings, hotels, etc., to have water wells drilled even though adequate and safe public water supply is available. Presumably, it is the privilege of such owners to develop and maintain such a semi-public supply if it is subjected to the same safeguards as are maintained for the public water supply. Unless the safety of the supply is definitely assured, however, the wells should not be drilled. Health departments having jurisdiction should have an opportunity, under such circumstances, to confer with the owner and to require him to adhere to proper standards of construction and operation. If such control can not be effected, the health department should have authority to prohibit the drilling of the well. In addition to securing a health department permit for drilling, it should be required that the owner advise the local water department, in writing, of his intention to have such a well drilled. This latter provision should also apply to private wells to be drilled where service from a public water supply system is available.

In Indiana, at present, the State Board of Health requires that plans and specifications for any major public water supply improvement be submitted for review and approval. In the case of the drilling of water wells, after the review of these documents has been completed, a well site inspection is made by a representative of the Board. After that, the Board has no further contact with the project until a water sample is submitted for analysis. After completion of the analysis, an opinion as to the quality of the water is expressed, but no further jurisdiction concerning proper capacity of the well, etc., is exercised.

Many municipal water department officials are in office for years without facing the necessity of carrying out water well drilling projects. Consequently, when a new well must be drilled, the water official can not necessarily be expected to have complete knowledge of the well drilling, developing, and testing methods. If the utility can not afford the services of a consulting engineer, the superintendent should, nevertheless, have available not only a comprehensive well drilling construction code or specifications, but also a health department to which he can turn for assistance. If general well drilling specifications, which would be applicable to all circumstances, were available, such health department assistance

might not be so important, but, since certain details must be worked out for each individual installation, it is desirable that the health department review the undertaking. Admittedly, such review is not necessary in many instances, but when regulations are drawn, they must apply to all alike.

It is believed desirable that a health department permit be required before a public or semi-public water well be drilled and that it also be required that the work be done by experienced, well-equipped drillers. Surely water works officials and engineers would not object to securing a well drilling permit, even though they might feel health department review of the project unnecessary in their case. On the other hand, perhaps the superintendent is as experienced in well drilling operations as the health department reviewing engineer. By submitting his plans to review, however, the experienced superintendent is not damaging his own standing in any way, nor is he yielding any right of "home-rule." Rather he is cooperating with the health board and perhaps, by his action, he may prevent his neighboring superintendent, who does not have extensive experience in such matters, from making a mistake which could result in heavy expense, loss of life, or, at least, embarrassment.

After a municipal water supply well has been drilled and developed, it is important that a report, including well log, pumpage test results, and other descriptive matter, be submitted to the state health department for review and for filing. In this report, conclusions as to proper pumping rates should be given so a conference with health department engineers can be held, if necessary, before establishing proposed pumpage rates. The filing of well logs with the health department may be well worth the cooperative effort, since, in later years, with changing local administrations, water department records may be misplaced and such information regarding underground formations lost.

In a number of states it now is required that well logs be filed with state geology or other similar boards. That is a logical procedure, but the requirement that public water supply well logs be filed with State Health Departments as well would also be reasonable, inasmuch as such information should be directly available to the health department when plans for extensions to the system are under consideration. In any event, it should be required that logs be filed with an administrative state agency so that they are available for reference by the general public.

Many other examples of benefits which would accrue from more

extensive control of water well drilling could be discussed, but it is believed that enough has been said to develop the fact that further control methods should be worked out.

It may appear that too great emphasis has been placed upon state health department control of well drilling. This has not been done through any desire to build up health department duties or prestige, but, rather, because it is recognized that even though other than health matters are involved, such other considerations are of relatively minor importance when compared with those affecting health. It is further recognized that state health departments already offer engineering review and assistance in connection with water supply problems and that, in general, there is no other public agency so well qualified to administer control over water well drilling.

It is believed proper, and in the public interest, that appropriate steps be taken to accomplish further control over water well drilling operations, and that this control can best be accomplished by the adoption of a water well drilling code setting forth minimum requirements with respect to well drilling operations and, if desirable in the given instance, requiring the licensing of water well drillers.

The standard well drilling specifications now being prepared by the A. W. W. A. might well be arranged in such a manner that they can, by reference, be used as a basis for minimum standards of water well construction and, thus, be suitable for incorporation in construction specifications or in well drilling codes to be adopted by supervisory agencies. In most states, health departments could legally promulgate water well drilling codes based upon standard specifications drawn by the A. W. W. A., but it would probably be difficult to enforce licensing provisions without direct statutory enactment.

In such codes it should be required that notice of intention to drill private and semi-public wells should be given to water department officials where public water supply facilities are available to the premises upon which the private wells are to be drilled. Also health department permits should be required before public and semi-public water supply wells may be drilled; and after drilling operations have been completed, filing of reports, including well logs, etc., should be required, together with the submission of water samples for sanitary analysis. Such permits need not be required when private wells are to be drilled, but, in all cases, it should be required that drilling operations be carried out by experienced and well-equipped water well drillers working under an official drilling code.

# Cleaning Wells in Parkersburg, W. Va.

*By Golden Underwood*

THE source of water supply for the City of Parkersburg, W. Va. consists of eighteen wells located along the east bank of the Ohio River, just north of the city limits. The wells are of the gravel packed type, with an outside casing of about 36 to 38 in. in diameter and an inside casing 18 in. in diameter. Attached to the inside casing is an 18-inch screen; and the space between the casings is packed with gravel.

The capacity of the wells, when drilled more than ten years ago, was about six million gallons per day. By 1939, this combined output had decreased about 50 per cent. Since the wells are located so close to the Ohio River, it was felt that the decrease must be caused by the clogging of the screen by corrosion, rather than by lowering of the water table. This belief was substantiated when the small screen on the bottom of the pump bowl was taken out for inspection.

To restore the lost capacity, it was decided to experiment with a new method of cleaning, as follows:

Ten carboys of muriatic acid (about the amount necessary to fill the screen) were poured into the well through a 3-inch pipe, fitted with a large funnel. After all the acid had been poured, steam was applied through a small pipe, which extended to within about 12 in. of the bottom of the well. The steaming, which was continued for from 4 to 6 hr., both warmed the acid and served as an agitator. Then, about 24 hr. after the application of the acid, from 1,000 to 1,200 lb. of soda ash were poured into the well (by a worker wearing a gas mask) and steam applied in the same manner as with the acid.

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A paper presented on October 26, 1940, at the West Virginia Section Meeting, Huntington, W. Va., by Golden Underwood, Water Commissioner, Parkersburg, W. Va.

Following this treatment, the well was permitted to stand for about a day. Then, a swab and sand bailer pump were used to remove the corrosive particles and sand that were present. Finally, the pump was replaced in the well and started, pumping water to waste over the river bank for about an hour before being reconnected with the service lines.

TABLE 1

*Test Results, Before and After Cleaning, on Well No. 8*

| BEFORE CLEANING | | AFTER CLEANING | |
|---|---|---|---|
| Yield | Drawdown | Yield | Drawdown |
| *g.p.m.* | *ft.* | *g.p.m.* | *ft.* |
| 190 | 12 | 405 | 12½ |
| 245 | 15 | 430 | 15½ |
| 275 | 18 | 456 | 19 |
| 300 | 21 | 475 | 20½ |

Results of the process, as shown in Table 1, were very satisfactory. In all, production was increased almost 60 per cent; and some individual cases showed much greater increase than this figure indicates. Well No. 9, for instance, tested at a maximum production of 110 g.p.m. before cleaning and at 400 g.p.m. after cleaning.

# The Bacterial Corrosion of Iron and Steel

*By T. D. Beckwith*

MANY factors induce deterioration of iron or steel pipe and the more commonly understood of these causes have already been discussed at length. An additional factor which is active in disintegrating iron or steel, when conditions are favorable, is comprised of certain groups of bacteria. Only relatively recently has knowledge of these organisms in relationship to corrosion come into our scope of experience.

In this paper will be discussed the bacterial corrosion of iron and steel pipe. Through the discussion it will become evident that iron and steel are subject at times to the effects of this additional series of destructive factors; but it should be pointed out that the ideal structural combination for pipe has not yet been discovered, since faults may be observed in all of our present formulas. Iron, steel, clay, cement and concrete, are each open to certain criticisms; and it must be remembered that iron and steel pipe, in certain geographic areas, have resisted deterioration for many decades.

When considering the question of corrosion of iron by the activities of bacteria, one tends to think only in terms of a process of reduction. Nevertheless, it may be shown that there is an additional mechanism of oxidation whereby iron may be corroded by means of the physiologic activities of certain types. Thus in the following discussion we shall give attention to two particular subdivisions: (1) reductional bacterial corrosion of iron; and (2) changes which result from oxidation by micro-organisms.

In 1895, Beijerinck (1), at Delft, became interested in why mud at the bottom of a swamp is ordinarily black. He found that the black deposit owes its color largely to sulfide of iron and that this

A paper presented on October 24, 1940, at the California Section Meeting, Los Angeles, by T. D. Beckwith, Professor of Bacteriology, University of California at Los Angeles.

sulfide is produced by formation of hydrogen sulfide under anaerobic conditions. His student, Van Delden (2), in 1904, succeeded in isolating the causative organisms—now known as *Microspira desulfuricans**—in pure culture, thus making it possible to study the germ carefully. Van Delden found the organisms in fresh, marine and brackish water and showed that there are certain strains which thrive at temperatures as high as 45°C. Wolzogen Kühr (3), in 1923, ascertained that the organisms are present in the soils of Holland as far as 70 feet beneath the surface. Moreover (4), he demonstrated that the work of Beijerinck and Van Delden could be applied directly to the question of corrosion of iron pipe in Holland. An hypothesis of similar import had been offered by Gaines (5) in 1910, to the effect that corrosion was caused by a process which was particularly evident in the swampy regions of the Netherlands. Whereas it was once believed that bacteria are found only a few feet below the surface, the work of Wolzogen Kühr gave an inkling of the fact that certain kinds of bacteria are present at unsuspected depths. Very recently, Issatchenko (6) demonstrated that the *Microspira* may be found as deep as 6,000 feet underground. In 1926, Bastin (7) demonstrated the presence of organisms in crude oil from relatively deep levels below the earth's surface. He showed that the *Microspira* existed in crude oil sampled from 1,500 feet underground, and, moreover, that they not only alter the chemical structure of surrounding waters at that great depth, but that they initiate certain changes in the sulfur content of the crude oil itself. With regard to bacterial reductional processes of iron, outstanding work has recently appeared in England, by Bunker (8, 9); and, in this country, Hadley (10, 11), in Philadelphia, has offered much additional information. Beckwith and Moser (12) have demonstrated that similar changes are responsible for some defects in the manufacture of paper.

This type of corrosion of iron and steel, which in large part is brought about by production of hydrogen sulfide in close contact with the metal, has proved itself to be of fundamental importance in relationship to lines which carry sewage, water, oil and gas. On such pipe lines, the changes are likely to appear on the outside of the

*See *Bergey's Manual of Determinative Bacteriology* (5th ed., 1939), p. 123. The organisms have been variously named: *Spirillum desulfuricans* Beijerinck (1895), *Microspira desulfuricans* Migula (1900), *Vibrio Desulfuricans* Weinberg, Nativelle and Prevot (1937) and *Sporovibrio* Starkey and *Sporovibrio desulfuricans* Starkey (1938).

pipe, since, as will be shown later, conditions for growth of the bacteria are at their optimum on the external surface. On the contrary, in certain iron or steel tanks used for storage of crude oil, this mechanism of corrosion is noted frequently on the inside of the container and particularly along its base. Such large tanks frequently are placed above ground so that the organisms may not proliferate on the metal on the outside. Nevertheless, in contact with the bottom of such

FIG. 1. Iron Inlet Pipe to Septic Tank; after 6 years' exposure to bacterial action

tanks, there is likely to be an organic accumulation, sealed inside perfectly by the overlying layers of crude oil; and the organic sludge is likely to ensure ideal conditions for growth of the *Microspira*.

Figure 1 shows a broken section of an iron pipe which had been in use but six years before failure. The conditions under which bacterial reductional processes of corrosion could take place, in this instance, were at an optimum, since it was the inlet pipe of a septic tank. The accumulation of corroded material on the surfaces of the

pipe showed the presence of 60 per cent iron—very high for such an area. (Hadley has found certain specimens in which the iron deposit was 34 per cent.) The source for the iron in the detritus in the inside and upon the outside of the pipe was the structure of the pipe itself.

Figure 2 shows bacterial cultures of the organisms which were the cause of the trouble in this instance. Those which appear black in the photograph, are the active cultures. By use of proper media, iron sulfide was caused to form in the tubes themselves. The third and lighter tube (right) is a control which was not inoculated.

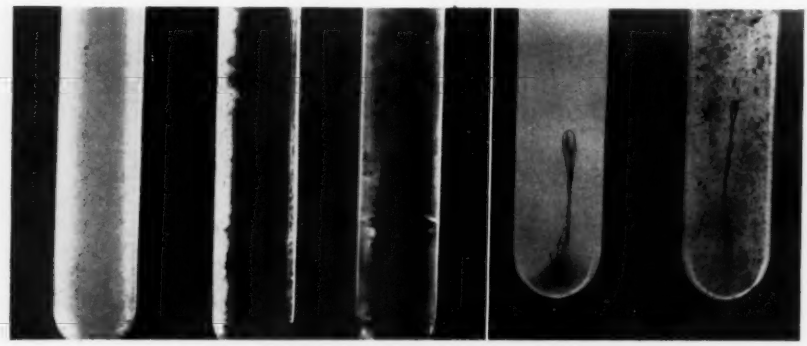

FIG. 2 FIG. 3

FIG. 2. Cultures of Organisms Which Destroyed Pipe (Tube at left is uninoculated control)

FIG. 3. Results of Experiment to Show Effect of Growth of *Microspira desulfuricans* in Formation of Soluble Compounds of Iron

## Characteristics of the Organisms

The characteristics, both morphologic and physiologic, of the organisms which bring about this destructive picture in iron and steel, when conditions are correct are particularly pertinent to the discussion. Morphologically, the organism, *Microspira desulfuricans*, is a curved rod and, thus, it may even appear somewhat comma-shaped. It is an anaerobe which thrives in the presence of a certain amount of sodium chloride or without sodium chloride; at as low as 10°C. temperature or, as high as 45°C. The present tendency is to consider that three supposedly different types, which are accustomed to the variety of conditions just noted, actually may be classed as but one type of organism which has become habituated to these varieties

of environmental influences. There is no agreement as to whether or not the organisms produce spores. Starkey (13) believes that spores are formed. In any event, it may be stated that the thermal death point may be relatively high.

Physiologically the bacteria are anaerobes. They utilize oxygen, deriving it, not directly from the free gas of the atmosphere, but by reduction of certain compounds in which the oxygen is contained molecularly. In the same way, they may utilize sulfate, sulfite, thiosulfate, or organic substances. Of these compounds, sulfates and organic compounds, containing sulfur and oxygen, are those most frequently found in soil. It is in soil that the organisms thrive and it is in soil that pipes are buried. The anaerobic conditions, which are essential to the growth of the bacteria, obtain in a swamp or in a poorly drained soil where oxygen tension is low, though it is not necessary that the soil be saturated completely with water. Thus, the *Microspira* are less frequently active in gravel, where there is better drainage than in clay or in silt.

Moisture is essential to the multiplication of the *Microspira* for two reasons: (1) it dissolves the food essential for the life processes of the bacteria and also dissolves their by-products; and (2) it tends to replace the oxygen in the soil and thus to intensify the anaerobic environment.

Sulfur is also essential to the organisms. It has already been noted that sulfates are commonly used by the bacteria. There are, however, many organic compounds which contain sulfur. Among them may be listed cystine, cysteine, and methionine which, with many other similar compounds, may be reduced in the formation of hydrogen sulfide. The hydrogen sulfide, in turn, combines, in part, with iron or with any other substance for which it may have chemical affinity.

The question of the hydrogen ion concentration of the soil solution is also important. The destruction of protein in soil is attended by the production of ammonia, a weak base, as one of the by-products. On the other hand, carbohydrates, such as cellulose, when broken down in soil, produce acid. Thus, in the natural destruction of organic matter in soil, there are formed a weak base and a weak acid. The resulting tendency is for the soil to become slightly basic unless flushed well by heavy rainfall.

This effect of the hydrogen ion concentration upon the growth of the organisms has been considered in the following experiment.

Media favorable to the organisms were made and then buffered to a hydrogen ion concentration range of 8.5, of 7.0 and of 5.0. These series of tubes were then inoculated with three crude cultures of the bacteria. Of the three crude preparations, two had been derived from irrigation waters in Southern California, and the third was isolated from rotting sewage. Incubation produced highly active growth in tubes at pH 7 and at 8.5, but little or no growth could be demonstrated at a pH of 5.0. Thus, it was indicated that the *Microspira* prefer a soil solution neutral or slightly basic. The lack of growth of the bacteria in media having a reaction of pH 5.0 need offer no encouragement to the engineer, since iron placed in a solution of similar acidity will corrode slowly due to combination of the acid with the metal.

Carbon, as a source of energy, is essential for the growth of the *Microspira*. Such carbon is easily found since in the organic matter of soil there is much cellulose, one of the most common components of all sorts of vegetable matter. In addition there may be other types of carbon-combining compounds, such as traces of starch or of certain mono- or disaccharides, present. The bacteria have been demonstrated by Baars (14) to be able to utilize carbon from many of the fractions resulting from disintegration of cellulose, from starch, or even from many lipoids. Thus, the essential carbon certainly will be present in any natural soil environment.

As has been stated previously, temperature is not a determining factor. Thus, the process which will become accelerated during summer months, because of optimum temperatures, will, nevertheless, not be suppressed entirely during the winter period, when temperatures are above the freezing point of water.

That the growth of the *Microspira* actually does give rise to products which attack iron has been proved by the following experiment. Sixty milligrams of pure iron wire were immersed in a soft deep agar culture of the crude organisms. Incubation then proceeded at 30°C. From time to time the iron was transferred to a fresh culture so that the metal, insofar as possible, might continue in contact with continuously multiplying bacteria. This experiment progressed during a period of six months, and, at the end of that time, it was found that the initial 60 mg. of iron had lost 21 mg. of weight.

There appear to be at least two possible methods of demonstrating the effects of this type of bacterial corrosion of iron or steel. The first of these has been given careful attention by Hadley (11). It

consists of applying, to the suspected area of metal, a small amount of dilute hydrochloric acid. If iron sulfide is present, the odor of hydrogen sulfide should become evident. It is deemed possible that complicating factors may interfere with the complete validity of this test, inasmuch as certain workers point to the fact that in iron there is naturally a small content of sulfur. Thus, it has been argued that the evolution of hydrogen sulfide following this treatment does not of necessity prove that the sulfide was formed by the activities of the bacteria. Nevertheless, the test is valuable.

The second method of detection depends upon cultural technique. Deep agar tubes may be prepared according to two different formulas. One of these contains sulfate and the other includes organically combined sulfur. In the second formula, the sulfur desired will be found in the structure of certain amino acids which are present in the peptone and which serve as one of the components of the medium. The formulas may be found in the work of Beckwith and Moser (12), but it should be stated that there are others of somewhat similar content which are likewise very usable. The components are compounded as follows:

*Medium for Sulfate Reduction*

| | |
|---|---|
| Potassium monohydrogen phosphate | 0.5 gram |
| Asparagin | 1.0 gram |
| Magnesium sulfate | 2.5 grams |
| Sodium lactate | 2.0 grams |
| Ferrous chloride | 0.1 gram |
| Tap water | 1,000 ml. |
| Agar | 20.0 grams |

*Medium for Reduction of Organic Sulfur*

| | |
|---|---|
| Peptone (Difco) | 20.0 grams |
| Ferric chloride | 0.1 gram |
| Tap water | 1,000 ml. |
| Agar | 20.0 grams |

Some of the suspected material should be mixed with the agar which previously has been melted and then cooled to approximately 50°C. The tube then must be layered with sterile "Vaspar," which contains equal parts of paraffin and vaseline, or with sterile vaseline itself. It is the function of this overlying seal to prevent access of atmospheric oxygen to the preparation below, since the organisms are anaerobes. In any test, it is wise to utilize tubes of both of the formulas mentioned. Incubation should proceed at approximately

30°C., and the test very likely must be prolonged. No test should be terminated until at least 30 days have elapsed. Presence of the organisms will be indicated by the appearance of areas of blackening in the tubes. Finally, smear preparations for microscopic examination of the contents of these blackened areas should be made.

Certain types of corrosion of iron or of steel tend to become self-limiting. It has been shown, however, that this is not true of the process of bacterial disintegration of iron or of steel. The curves which have been plotted maintain much steepness as the process proceeds toward termination.

An interesting theory, advanced by Wolzogen Kühr and Van Der Vlught (15), indicates that not all of the effects of disintegration of these bacteria upon the iron are due to the hydrogen sulfide alone. The authors have presented certain chemical equations by means of which it may be shown that hydroxyl ions are formed and that these, in turn, may combine with the iron to produce the hydroxide, probably ferrous. Thus, additional loss of iron results. Whereas the iron sulfide produced in the reaction is insoluble in water, the ferrous hydroxide is soluble. The result, then, will be that some of the iron-containing products of deterioration of the iron of the pipe may be demonstrated at some measurable distance from the pipe itself. Therefore, iron sulfides in contact with the pipe do not furnish a complete index of the degree of corrosion.

In connection with this theory of Wolzogen Kühr, it was desired, if possible, to demonstrate the effect under laboratory conditions. The results, shown in Fig. 3 (see p. 150), may be described as follows:

The formula of the sulfate-containing medium, which has been listed earlier in this discussion, was compounded, but to that particular batch no trace of iron was added. The medium was then placed in a large test tube on the bottom of which rested a small amount of iron wire. The agar contents of the tube were then inoculated with a crude culture of the *Microspira* which had been isolated from decomposing sewage. After layering with the "Vaspar" seal, the tube was incubated for approximately two months at 30°C. Early in the test, by magnification with a hand lens, one could observe the growth of the colorless colonies of *Microspira* within the medium, but early in the experiment, they could show no black color, because there was, then, no iron in the solution. Nevertheless, as the experiment progressed, the colonies began to blacken. The hydrogen sulfide formed by the growing organisms, since it was

soluble in water, could pass to the area where the iron was immersed and there a certain amount of insoluble iron sulfide was produced. Other portions of the iron, however, were placed in solution in the tube, as may be demonstrated by reference to the chemical equations presented by Wolzogen Kühr. The soluble salts of iron, in turn, diffused from the iron to the area where the colonies were growing; and, as a result, in that area adjacent to the colonies, blackened zones appeared ultimately. In Fig. 3, then, it will be noted that colonies, at a distance from the iron which is visible in the bottom of the tube, became blackened. When the experiment was terminated, the tube was broken and the odor of hydrogen sulfide gas became evident instantly. Moreover, when some of the agar from the upper portions of the culture, far removed from the iron, was macerated with distilled water and a test made for iron in solution, a very positive result developed. Thus, it has been possible to present a visual demonstration of the results which have been indicated by the somewhat abstruse chemical applications of Wolzogen Kühr; and it is shown that this additional effect upon the iron both amplifies and complicates the picture.

Methods for the prevention of this type of corrosion have not yet been developed in satisfactory degree. One may derive certain benefits, however, by consideration of the following points:

If the soil is well drained, attacks upon the pipe will be less than those encountered in a swampy area. In certain regions, as in Holland, attempts have been made to lay iron pipe in gravel as much as possible. In certain select terrains, such procedure may be feasible, but in many other areas, the use of gravel around a pipe can produce no benefit, as, for example, in a swamp.

Engineers have given attention to resistant coverings for iron or steel pipe, but this problem likewise remains unanswered. The covering must not under any condition be of such nature that it can produce a metallic couple with the iron. At the same time, it must have a strong bond with the metal, and one which does not break nor rupture either by expansion or by contraction. It must be sufficiently rugged to withstand pressures of soil and it must not be subject to bacterial or chemical disintegration. Attempts have been made to utilize cement, asphaltum, and other crude oil derivatives, such as tar. Attention has also been given to plastics. The question of expense for covering becomes very important. It is to be hoped that further discoveries may present the engineer with a

really satisfactory covering material which can meet all of the specifications enumerated.

If the pipes were placed aboveground, certain benefits would be derived, but there are many reasons why this would not be feasible. Pipe lines raised above the surface of the ground would offer definite commercial, military and esthetic difficulties; and, in addition, the question of added expense would constitute a material factor.

### Experiments on Cathodic Protection

Cathodic protection against the attacks of bacteria for metal pipe has received serious thought by certain authorities, and, at the very least, it may be stated that its use for inhibition of this form of corrosion is controversial. This type of protection has proved of undoubted value against electrolysis, but electrolysis and bacterial attack differ fundamentally. In cathodic protection, the desire is to polarize the iron by causing it to be covered with a thin film of hydrogen—an effect somewhat reminiscent of the polarization which may appear in a battery. Since, however, the *Microspira* are anaerobes, and since the presence of hydrogen infers that the tension of oxygen is diminished markedly, the *Microspira*, which use hydrogen as an acceptor of certain other products of disintegration, will thrive when surrounded by an atmosphere of hydrogen, if food essentials are available.

The following laboratory experiments, dealing with the outcome of cathodic protection, have been conducted by the writer, using deep, thin agar cultures in large test tubes of approximately $1\frac{1}{4}$ in. diameter. The medium was that in which reduction of sulfate is tested, the one difference from the formula given earlier being that no iron salt whatever was added when the media were compounded. The deep tubes were sowed with a crude culture of the *Microspira* and, then, when the agar had become semisolid, an 18-gage platinum wire was thrust down to the lower curvature and against the glass wall of the tube. Opposite the platinum wire, a 25-gage iron wire was inserted and placed against the glass wall diametrically opposite the platinum. The tubes were then layered with "Vaspar" and the electrodes were connected to a supply of direct current delivering $1\frac{1}{2}$ volts. In certain of the tubes the current passed from the platinum to the iron electrode, while in others the direction of current was reversed. The tubes, set up in this manner, were incubated at 30–37°C. for a period of ten days, during which time the current was

constantly in flow. Three sets of experiments were carried to completion. It was noted that when the current passed from the iron to the platinum, the agar darkened promptly; and at the termination of the test, a black deposit of the iron and of iron compounds was found on the surface of the platinum. Within six or seven days, the iron of the electrode had disappeared. Microscopic examination of the agar suspension from the vicinity of the iron and platinum electrodes showed many of the *Microspira;* and a test for iron in solution, made by macerating some of the agar from the vicinity of each electrode in distilled water, yielded a positive test in each instance. In turn, sub-cultures made from the agar, taken from the vicinity of each electrode, gave positive growth of the bacteria in all instances. Thus, the viability of the organisms was proved.

The real interest of the experiment, however, lay in the outcome of the tests when the current moved from the platinum to the iron electrode; for if hydrogen can protect the metal from attacks by the bacteria, little or no change should have appeared on the iron. Nevertheless, in one instance, one iron electrode had corroded entirely to fragmentary condition in ten days. In the other two instances, the iron wire was intact at the close of the ten days' period of incubation, but it was so badly corroded that three or four more days of exposure would assuredly have also disintegrated these electrodes. Meanwhile, the agar throughout the tube had darkened because of the formation of ferrous by-products. Bacterial smears made from the agar in the immediate vicinity of each electrode showed many *Microspira* in the microscopic field. Sub-cultures, in all instances, gave positive growth, showing that the bacteria were alive; and when agar from the area of each electrode was macerated in distilled water, a positive test for iron resulted. The odor of hydrogen sulfide was present in all tubes.

As a control for the experiments, an uninoculated tube of the medium was set up. In the tube, a length of the iron wire was inserted in the agar, but no electrical connection was included. After ten days incubation, no visible change had appeared on the iron.

Thus, these laboratory experiments, attempting to show that cathodic protection does interfere with the corrosive effects of *Microspira*, gave a negative result in each of the three instances.

This question of the possible efficiency of cathodic protection against bacterial corrosion is of such importance that even though

the laboratory tests gave a negative outcome, the project should be examined with care, under field conditions, by competent engineers and trained bacteriologists working in the closest cooperation. It is possible that such field experience may yield an outcome differing substantially from that indicated in the laboratory.

### The Rôle of Oxidation of Sulfur

Attention to the possible bacterial attack upon iron by oxidation of sulfur arose from certain observations made in 1931 and reported

FIG. 4. Portion of a Pipe Line; showing bacterial attack upon iron by oxidation of sulfur

by Beckwith and Bovard (16). At that time, attention was drawn to a certain iron pipe line which had been constructed with a sulfur-containing joint-sealing compound in the connections. The soil through which the pipe line passed was moderately well drained in many areas. Within approximately one year after construction, the line failed because of excessive leakage at the joints. Examination showed that the sulfur of the sealing compound had, in large part, disappeared. In the near vicinity, where the sulfur sealing compound had been applied, the pipe was definitely roughened. Bacterial cultures indicated that the sulfur had been oxidized by

bacterial action. It became evident, therefore, that not only had the sulfur-sealing compound failed in this particular set of circumstances, but that some by-product, formed during the disintegration of the sulfur, had, in turn, attacked the surface of the iron pipe itself in the vicinity of the joints. A portion of this pipe line at a joint is shown in Fig. 4.

Much work dealing with the oxidation of sulfur by bacteria of the soil flora had already been done by Waksman (17) and his collaborators. It was found that the by-product, sulfuric acid, will, in turn, combine instantly with any basic substance which may be available. This oxidation of sulfur may be induced by five or six varieties of soil micro-organisms. The various forms, as they grow in contact with sulfur, under laboratory conditions, give rise to an acidity, in terms of hydrogen ion concentration, which is nearly incredible to a biologist. It has been noted repeatedly that in certain culture flasks, a terminal pH as high as 0.8 has been produced. This is the approximate equivalent of 0.12 normal sulfuric acid. Not only do the organisms produce this intense concentration of acidity, but they live in the cultures for a considerable period of time. Other laboratory workers have discovered certain strains of these bacteria which have produced hydrogen ion concentrations as high as 0.6. The bacteria are aerobes, requiring atmospheric oxygen, but they form no spores. They multiply in an open or well drained soil, provided there is sulfur present. The sulfur frequently, under natural conditions, is obtained from disintegrated organic matter. Conversely, then, they do not multiply in a badly drained or in water-logged soil. On the other hand, although, for multiplication, they do require the conditions which have just been outlined, they will survive for considerable periods of time in locations where free atmospheric oxygen is almost lacking.

For methods of detection and isolation of the *Thiobacilli** one should refer to Waksman (18), who has presented a series of formulas which are very valuable in inducing the multiplication of these autotrophic bacteria. A small portion of the suspected material is placed in the fluid medium and then is incubated at approximately 30°C. for a period of time which, in length, will depend upon changes of hydrogen ion concentration, which are usually demonstrated by means of the

* See *Bergey's Manual of Determinative Bacteriology* (5th ed., 1939), pp. 81–86, for list of eight species of the genus *Thiobacillus*, all deriving their life energy from oxidation of sulfur or sulfur compounds.

quinhydrone or the glass electrode. When a very distinct increase in acidity has been observed it has been the custom to sub-culture Waksman's agar medium and, then, when growth in colonial form became evident, to transfer back to the fluid medium and to observe resulting changes in hydrogen ion concentration in this flask under conditions of pure culture. Experience shows that the following formulas are valuable for these tests:

*Waksman's Medium No. 5*

| | | | |
|---|---|---|---|
| Distilled water | 1,000 ml. | $MgSO_4 \cdot 7H_2O$ | 0.5 gram |
| $(NH_4)_2SO_4$ | 2.0 grams | $FeSO_4$ | 0.01 gram |
| $K_2HPO_4$ | 1.0 gram | Sulfur | 10.0 grams |
| KCl | 0.5 gram | $Ca_3(PO_4)_2$ | 10.0 grams |

*Waksman's Medium No. 6*

| | | | |
|---|---|---|---|
| Distilled water | 1,000 ml. | $FeSO_4$ | Trace |
| $(NH_4)_2SO_4$ | 0.2 gram | $KH_2PO_4$ | 3.0 grams |
| $MgSO_4 \cdot 7H_2O$ | 0.5 gram | Sulfur | 10.0 grams |
| $CaCl_2$ | 0.25 gram | | |

*Waksman's Agar Medium*

| | | | |
|---|---|---|---|
| Distilled water | 1,000 ml. | $CaCl_2$ | 0.25 gram |
| $Na_2S_2O_3$ | 5.0 grams | $KH_2PO_4$ | 3.0 grams |
| $NH_4Cl$ | 0.1 gram | Agar | 20.0 grams |

The fluid media should be used in Erlenmeyer flasks which are to be filled to the level of largest area of the meniscus. Since the micro-organisms sought are aerobes, maximum expanse of the medium in contact with the air is indicated. Sterilization may be accomplished in an autoclave.

In the bacteriological laboratories at the University of California, tests to determine whether oxidation of sulfur by bacterial means can produce a corrosive action upon iron have been set up. Field observations already had indicated that such corrosive action is produced. Certain of the media recommended by Waksman for culture of the *Thiobacilli* were prepared. During the course of one experiment, a clean iron nail was placed in a flask of this fluid, and the flask, after sterilization, was inoculated with a very small amount of soil known to contain the organisms. Within seven days, the iron nail showed both discoloration and definite roughening of the surface; and subculture from the preparation demonstrated the presence of the *Thiobacilli*. An uninoculated control preparation did not show such alteration.

A quantitative test, as follows, was then run. To a flask contain-

ing formula No. 5 of the Waksman preparations were added 122 mg. of iron wire of excellent quality and into a second flask were placed 123 mg. of iron wire. The first flask was then inoculated by the addition of a small amount of soil which, previously, had been proved to contain *Thiobacilli*. The second flask was retained as an uninoculated control. These flasks were then incubated at a temperature of 28–30°C. for a period of two and one-half months. Early, it was noted that the contents of the inoculated flask were becoming definitely brown, while no color was evident in the control. Hydrogen ion readings were made on both flasks at the beginning of the experiment. In each instance, the degree of acidity was ascertained to be pH 5.5. At the end of the incubation period the inoculated flask showed a pH of 3.5, but a similar reading of the control flask indicated no alteration. A weighing of the iron wire in the inoculated flask revealed a drop from 122 mg. to 41 mg. of iron, or a reduction of two-thirds, but in the control flask there was no appreciable loss of the iron. Moreover, at the termination of the experiment, when the fluid contents of the test flask were submitted to the qualitative test for iron, the results were positive; but from the control flask, the results were negative. The two flasks are shown in Fig. 5. It was demonstrated, therefore, that the oxidation of sulfur which gives rise to sulfuric acid, when conducted in the presence of iron, results in solution of some of that metal in the immediate vicinity.

In Fig. 1 is presented the broken end of a section of iron pipe which had been corroded heavily through bacterial action. Not only did these products of corrosion both inside and outside the pipe show 60 per cent iron content, but, in addition, it was proved by chemical analysis that there was also 17 per cent of free sulfur. No sulfur-containing joint-sealing preparation had been used so the only possible source of this high sulfur content was the organic matter which had passed through the pipe. It is already known that the disintegration of organic matter is attended by evolution of hydrogen sulfide. At first glance, the mechanism whereby hydrogen sulfide in such an environment may give rise to the appearance of free sulfur is somewhat confusing. It will be remembered, however, that, in 1888, Winogradsky (19) demonstrated, by an impressive series of experiments, that there are several varieties of anaerobic bacteria which, during the course of their life processes, oxidize hydrogen sulfide to sulfur. One may assume, therefore, that Winogradsky's observations apply to this environment likewise; and that by the mechanism described by him, sulfur appeared. There is, therefore, a

second source of elemental sulfur in some soils which, in turn, may be oxidized by *Thiobacilli* if and as environmental conditions become correct for the growth of the sulfur-oxidizing bacteria. A condition such as the following may readily be visualized:

During the course of a rainy period, ground water has risen around the pipe line. This water, in turn, has brought with it much organic matter. Thus, at levels under the surface of the water, hydrogen sulfide is formed in the close vicinity of the pipe line. Some of the hydrogen sulfide will attack the iron to form not only the sulfide of

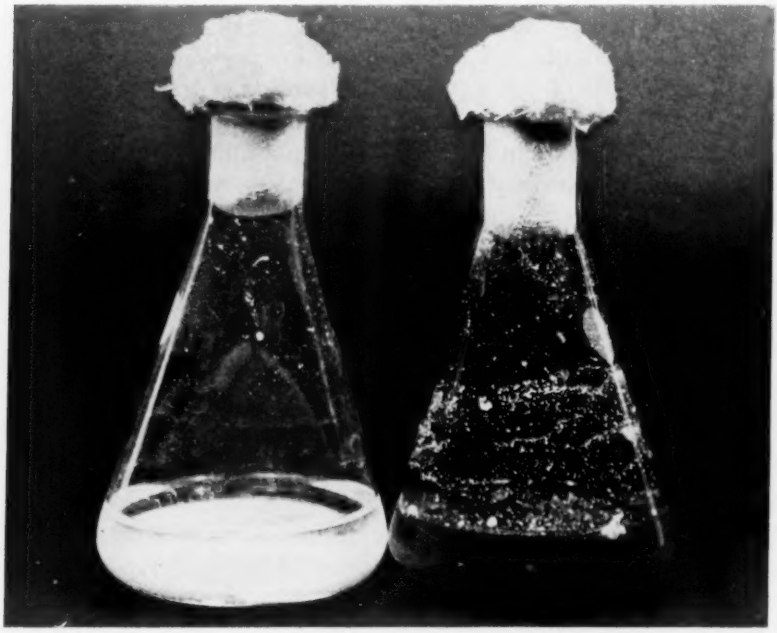

FIG. 5. Results of Quantitative Test to Determine Bacterial Attack Upon Iron by Oxidation of Sulfur (Flask at left is uninoculated control)

iron but also hydroxide. Still other portions of the hydrogen sulfide will be attacked by the bacteria, described by Winogradsky, to produce sulfur. So long as conditions remain anaerobic, that sulfur may be expected to remain inert, but later, when the water table drops and air re-enters the soil, with free sulfur available, *Thiobacilli* may be expected to recommence their multiplication and, in turn, to oxidize the sulfur, with the resultant formation of sulfuric acid.

To prove these assumptions experimentally, tests were set up to ascertain whether *Thiobacilli* actually were present and viable in the products of corrosion of the pipe which is pictured in Fig. 1. The anaerobic content of the material has already been described. Waksman's medium No. 5 was placed in a series of flasks, two of which were inoculated with small portions of the corroded material from the pipe, listed as samples Nos. 1 and 2 in Table 1. Sample No. 3 was removed from another similar section of iron pipe which had disintegrated after a number of years' use in an entirely different geographic locality. A fourth uninoculated flask was set aside as a control. From time to time, the hydrogen ion concentration in these flasks was observed; and then, when the experiment was terminated, the contents of each flask were streaked to plates of the agar medium recommended by Waksman for isolation of pure cultures. This last step is to be considered as confirmation. Reference

TABLE 1

*Activity of Sulfur-Oxidizing Bacteria Isolated From Products of Corrosion of Iron Pipe to Form Sulfuric Acid From Sulfur*

| DATE | SAMPLE NO. 1 | SAMPLE NO. 2 | SAMPLE NO. 3 | UNINOCULATED CONTROL |
|---|---|---|---|---|
| Oct. 14 | 6.6 | 6.6 | 6.6 | 6.6 |
| Oct. 21 | 6.0 | 6.1 | 5.7 | 6.6 |
| Nov. 1 | 5.2 | 5.2 | 4.5 | 6.4 |

to Table 1 will indicate that *Thiobacilli* were present and potentially active in each of the three samples tested. This determination of the presence of *Thiobacilli* in organic content of sewage is not presented here as an initial discovery, inasmuch as similar isolations were described, in 1929, by Ayyar and Norris (20).

In certain soils there is yet another source of sulfur, iron pyrites, which may be oxidized by *Thiobacilli*, with the resultant formation of sulfuric acid. Rudolfs (21) demonstrated that this mineral may be oxidized by bacterial action with formation of sulfate; and this change has also been tested under laboratory conditions. Certain flasks of Waksman's formula No. 5 were set up with the change of one item—the substitution of an equivalent amount, in terms of sulfur content, of finely ground iron pyrites for the sulfur designated in the Waksman formula. The flasks were then inoculated with a certain small amount of soil known to contain the *Thiobacilli*.

Within three weeks, the hydrogen concentration had changed from 6.2 to 3.8, although an uninoculated control flask did not reveal any variation of acidity. Attention has been drawn to the importance of iron pyrites as a source from which sulfuric acid may be formed biologically in the instance of an iron pipe line from a region rich in the formation and through which drainage waters have been carried. The pipe line in question suffered much corrosion.

Very recently, Wolzogen Kühr (22) has called attention to yet another mechanism in soil, whereby sulfuric acid may be liberated when certain reactions involving iron sulfate take effect. These reactions may possibly be demonstrated to be of fundamental importance by future experimentation.

It has been the writer's endeavor, during the course of this discussion, to show that corrosion of iron or steel pipe may be carried on biologically by a series of mechanisms. Not only is iron sulfide formed but soluble compounds of iron are produced in the process of reduction. This process of reduction frequently is considered to comprise the only mode by means of which iron or steel is disintegrated bacterially. Also, he has endeavored to demonstrate that there is an additional process, oxidation, that is functional by means of the activities of other groups of bacteria. This series of processes is interlocking, and, as environmental conditions change, the mode of attack upon the iron also may alter. In any event, iron or steel, in immediate contact with these various forms of growing bacteria, may suffer attack.

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**Discussion by *C. K. Calvert.*** * The paper by Dr. Beckwith is most interesting in subject matter and splendidly written. It should be studied by all who handle water.

In the majority of distribution systems, actual physical failure of mains is rare, but bacterial growth in them may be of tremendous importance from the standpoint of taste and odor.

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* Superintendent of Purification, Indianapolis Water Co., Indianapolis, Ind.

Few failures have occurred in mains in Indianapolis. *Microspira desulfuricans*, however, has been found in raw water and in samples from dead-end mains. The density of the organisms increases as the free oxygen content falls. In experimental dead-end lines, the organisms were more prevalent when sterilized hemp was used as a joint material than when lead wool was used. Apparently, the hemp is a source of food.

The *Microspira* is found occasionally under a slime growth made up of capsulated organisms. In one case a cast-iron anti-frost meter bottom on a domestic service developed a leak. When the slime growth on the bottom was removed, under the copper plating, in which there was a minute hole, the iron was graphited and yielded *Microspira* when planted in Starkey's lactate broth (Arch. f. Microbiol., **9:** 268 (1938)).

To be able to transfer cultures, it has been found convenient to grow them in bottles or tubes filled with nitrogen and closed with tight-fitting sani-tab rubber caps. Transferring cultures from tubes sealed with oil or wax offers some difficulty.

While *Microspira desulfuricans* is wide-spread and has been found repeatedly, it is not demonstrated that it, alone, is responsible for taste and odor development in dead ends. It appears, from the small amount of work done to date, that symbiosis or synergism may play a part, and that a number of organisms, not growing on ordinary media, may be involved. In this connection, the article by Alexander (Jour. A. W. W. A., **32:** 1137 (1940)) is of special interest.

This whole matter has many possible ramifications and Dr. Beckwith has made a valuable contribution to the work on the subject.

**Discussion by *Max Suter*.*** Prof. Beckwith brings forth, in a clear way and with interesting methods of research, the two types of corrosive action of sulfur bacteria (using this expression in the broad sense of all organisms that handle some form of inorganic sulfur as a main part of their metabolism). One type lives anaerobically and produces hydrogen sulfide by reduction. The other type, living aerobically, produces sulfuric acid by oxidation. Both bacterial end-products cause corrosion of cast iron and steel. Under varying outside conditions these processes may follow each other, a sequence which is technically disturbing because remedies against one type may favor the growth of the other type.

* Engineer, State Water Survey, Urbana, Ill.

It has, however, to be considered that this investigation has not been made with pure cultures and that therefore symbiotic effects, which may influence mainly the quantitative results, may be active

The action of sulfur bacteria is not the only phase of bacterial corrosion of iron and steel, as the title might imply. Buswell and Suter (Jour. Bact. **39:** 583 (1940)) have shown that a sludge can be formed by organisms oxidizing ammonia to nitrite and (in an unpublished paper given before the Kansas City Convention of the A. W. W. A., 1940) also by those that oxidize nitrites to nitrates. In these sludges are found large amounts of inorganic ferric hydroxide, although the water pumped into the mains is practically free of iron. It is very probable that the nitrous and nitric acids produced in these actions corrode the pipes under the slimy sludge and cause the formation of the ferric hydroxide under the influence of additional ammonia.

In most cases, the sulfur bacteria may act on the surface of the pipe, whereas the nitrogen organisms were found active on the inside. There is, however, no reason why the reverse location may not happen under certain favorable conditions.

A very interesting property of both these groups of organisms is their slow growth. The additional behavior to be considered is that they do not readily develop on standard media but require, for their detection, a procedure that is to some extent complicated. This limits to special occasions the routine application of the tests for the determination of their presence.

The existence of bacteriological corrosion by sulfur and nitrogen organisms shows that water bacteriology is really more than just the total count and lactose fermentation. In fact, many other fields of bacteriology play an important role in the full picture of single cellular life in water. To mention only a few: We certainly need more knowledge of the types of organisms found in the total count, of those that give a false presumptive test and of the chlorine-resistant bacteria. Tests for many pathogenic organisms are still a gamble. Autotropic organisms, such as the iron, sulfur, nitrogen and methane users are really little known. The role of bacteriophage is more suspected than proven and symbiotic actions have received little attention. It is hoped that these other phases of water bacteriology may soon receive as much study as is showered on the test for coliform bacteria.

# Control of Iron Bacteria by Super-Chlorination

*By E. B. Carroll*

THE experience with taste and odor to be discussed here was caused by *Crenothrix* in the water supply of the Federal Homestead Project at Arthurdale, West Va. This supply is small and furnishes water to only part of the homes on the project, the daily consumption being 14,000 gal. Twenty homes, five school buildings, one inn and the Community Center are served.

The water is obtained from three wells. One is 739 ft. deep and is located along the highway near the Community Center. The other two are much shallower, only 86 ft. and 118 ft. deep, respectively, and are located near the school buildings.

The two shallow wells were intended for use as an emergency supply, but they have been used almost constantly. The supply from them is pumped into the line feeding the school buildings, the reservoir acting as a float.

The deep well is pumped directly into the 50,000-gallon reservoir and the water is fed by gravity to the homes, inn and the Community Center. The only treatment is chlorination with hypochlorite solutions measured through a W&T Chloro-Clock.

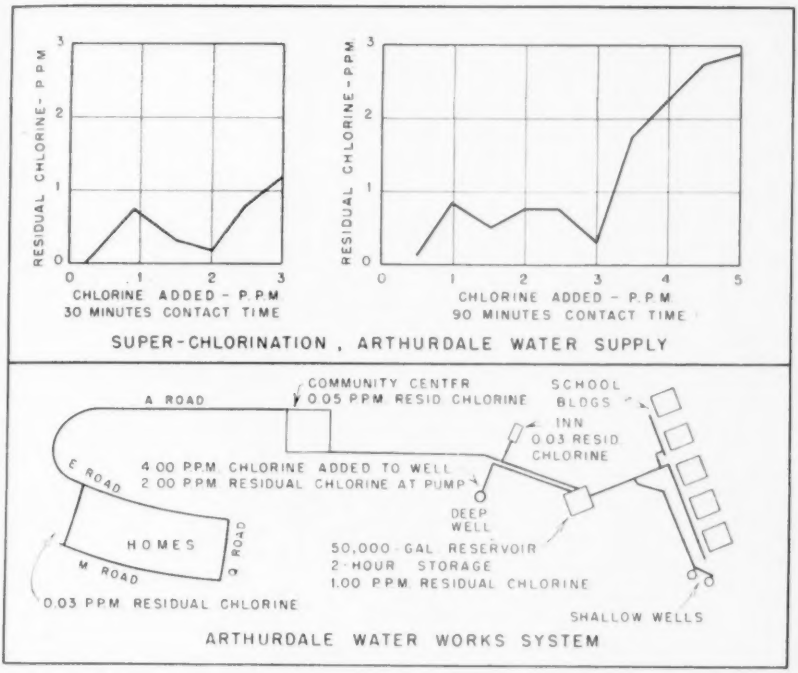
In the fall of 1938, a disagreeable musty odor was noticed in that part of the system furnishing water to the inn and homes. This condition increased until the water had an odor and taste similar to decayed vegetation; and when the ends of the lines were drained, the water was found to be black.

Assistance was requested from the Public Health Training Center; and in its laboratory analysis it was determined that the condition was due to the growth of *Crenothrix*, a filamentous organism, one of

A paper presented on October 25, 1940, at the West Virginia Section Meeting, Huntington, W. Va., by E. B. Carroll, Sanitarian, Preston County Health Department, Kingwood, W. Va.

the characteristics of which is the formation of brownish masses or flocs. The reservoir had a half-inch deposit on the walls and the top of the water was completely covered.

In the early spring of 1939, the reservoir was drained and scrubbed with a solution of chlorine and copper sulfate and the well was treated several times with heavy doses of copper sulfate. After each of these treatments, a large amount of black sludge was pumped to waste from the well.




After treating the well and cleaning the reservoir, ammonium sulfate was added in the treatment to maintain a chloramine residual through the reservoir and, if possible, into the distribution system. With a chlorine residual as high as 2.00 p.p.m. at the pump, it was never possible to get more than a trace (0.03 to 0.05 p.p.m.) of chlorine into the system. The treatment, however, reduced the taste and odor until it was noticeable only in the hot water; and except for two unexplainable recurrences of the disagreeable musty taste and odor during the summer, this condition was maintained.

In October, 1939, after publication of the new technique of super-chlorination, a laboratory test on the water from the deep well was made to determine the proper super-chlorination dosage. On November 26, super-chlorination, based on the laboratory results, was initiated.

Instead of de-chlorinating, ammonia was added at the reservoir (75 minutes after chlorination) to convert the residual chlorine to chloramines. Starting with a dose of 9.00 p.p.m., a residual of 2.00 p.p.m. (chloramine) quickly spread through the system. This caused such a strong chlorine odor that the dosage had to be reduced to 4.00 p.p.m.

In May, when it became necessary to remove the tubing from the well for repair, preparations were made to add the chlorine directly at the well.

At the present time, with the application of 4.00 p.p.m. chlorine, the residual at the pump is 2.00 p.p.m. and at the reservoir, after 75 minutes contact, 1.00 p.p.m. It should be noted that not liquid chlorine, but high test hypochlorites are being used in the treatment, and that no form of ammonia is now being added. After the water flows through the reservoir, which has more than 72 hours theoretical detention capacity, the chlorine residual in the distributed water is 0.03 p.p.m.

The treatment resulted in satisfactory conditions during the summer of 1940 and has continued to do so up to the time of writing (October, 1940). The growths did not develop on the reservoir walls during the summer and the distribution system problems appear to have been controlled.

# ABSTRACTS OF WATER WORKS LITERATURE

**Key.** **31:** 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31:** 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, *[ ]* is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene (British); C. A. —Chemical Abstracts; P. H. E. A.—Public Health Engineering Abstracts; W. P. R. Water Pollution Research (British); I. M. —Institute of Metal (British).*

## DEFENSE

**Preparedness in Water Supply.** CHARLES HAYDOCK. Am. City. **55:** 8: 56 (Aug. '40). No modern city can exist without adequate water supply; hence water works industry occupies singular position of responsibility and importance in preparedness. Water works supt. must be alert to weaknesses in system under his control and strengthen them. Author propounds many questions which if adequately answered will produce water systems more immune to successful attack.—*Arthur P. Miller.*

**Sabotage of Water Works.** ANON. Am. City. **55:** 12: 35 (Dec. '40). Purposes of sabotage are to incapacitate people and destroy or retard their activities, both of which might have serious effect upon normal living and production and thereby further ends of enemy power. Can be done by direct agents of such power or by others, in pay of such agents, who wish no good to those sabotaged. Other acts committed by persons for spite or for retaliation against real or imagined ill done to them cannot be overlooked. In municipal or industrial water system, weak points are machinery, gate houses and dams, pipes crossing rivers or bridges, power lines, all pumps, ungated intersections in high-value areas and distribution reservoirs where disinfection is not directly available. Pre-organization and establishment of unified, group thinking are important in combating sabotage. Fingerprinting of employees (concerning which honest have no cause to worry) and careful investigation of trustworthiness of employees should be undertaken. Protective fencing, flood lighting, patroling, concealment of important fixtures, installation of alternate control devices and those making for distant control, and accumulation of ample supply of equipment to permit replacement are all worthy of consideration.—*Arthur P. Miller.*

**Sabotage and Water Service.** ANON. Eng. News-Rec. **125:** 86 (July 18, '40). Summary of information obtained in canvass of water works authorities in key industrial cities, national agencies concerned with defense activities, equipment manufacturers and consulting engineers, regarding measures that should be taken to safeguard water service from sabotage. Municipality should assign responsibility for safeguarding water supply system to competent, trustworthy individual or to small committee selected from men in water dept. Precautionary measures fall into 4 categories: (1) those of general significance, (2) safeguards to source of supply, such as dams, intakes and aqueducts, (3) safeguards to treatment works, and (3) elimination of distr. system weaknesses. One large city has made thorough check of affiliations and attitudes of all its em-

ployees. Others have instructed employees to report all unusual incidents and to be on lookout for strangers. Other suggestions are that summary of water resources be prepared in convenient form, stocks of repair and emergency materials built up, consideration given to installation of standby units, and plans for emergency service formulated. Protection of dams, intakes and aqueducts is difficult. Principal precautions at dams and intakes have been to curtail fishing. It would be practical to restrict operation of airplanes in vicinity of dams and reservoirs. One city is providing floodlighting at its intake. Consideration should be given to substitute facilities for obtaining raw water if normal service is disrupted. Parking should be prohibited on highways and streets in vicinity of dams and other structures. Aqueducts and supply mains are, perhaps, most vulnerable point in system. One city has lowered access manholes to tunnel below ground level and sodded surface over them, noting, of course, their location. Agreed that visitors should be banned from water works properties at present time. In some cases, properties have been fenced. Impairment of water quality is possible but not probable. Few, if any, attempts were made during World War to poison water even in combatant areas. More frequent sampling, however, is advisable. Reported that Paris planned to add harmless blue or green dye to supply if contamination was discovered. Maintenance of reserve supplies of chlorine and portable emergency chlorination equipment would be advisable. Must be remembered, however, that chlorine itself might be used with criminal intent and should, therefore, be kept in safe place. Precautionary measures in regard to distr. system are chiefly extension of normal improvement measures. Adequate valving so that sections may be isolated and interconnecting mains so that all areas can be served from 2 or more points are important. Well organized emergency crew is primary defense measure. Relationship of Federal Bureau of Investigation and the National Guard to protection has not been publicly defined, but it is logical to assume that advisory service of these organizations is available to water works authorities.—*R. E. Thompson.*

**Watch Well Supplies.** DAVID G. THOMPSON. Eng. News-Rec. **125:** 454 (Oct. 10, '40). Some possibilities of sabotage of ground-water supplies are pointed out and min. precautions are stated to be housing of wells and pumps in substantial structures, floodlighting and, perhaps, fencing. In many places, material should be available for making cross-connections to industrial wells in case of emergency. Adequacy of supply should be constantly considered.—*R. E. Thompson.*

**A.R.P. [Air Raid Precaution] and Water Undertakings.** GRANVILLE BERRY AND ALAN BOOTHMAN. Surveyor. (Br.) **98:** 83 (Aug. 16, '40). Cooperation of suppliers has been in operation for considerable time with conspicuous success from standpoint of supply and quality of water. If some of wider projects, raised within recent years, for tying in systems of large towns had come to fruition, safety of suppliers over a wide area would have been made secure against possibility of serious damage to sources of supply. Whenever possible A.R.P. scheme of water undertaking should include effective system of mutual assistance. Appeals for assistance would of necessity have to be governed by imminence of attack in assisting authority. Result of organized system of waste detection and elimination will be distinct contribution to effort of water service to meet obligation if and when need arises. Redistribution of population is added reason why fullest advantage should be taken of realizing max. capac. of existing mains. Main problem must necessarily be to obtain sufficient water for fire-fighting purposes in emergency. A tendency in recent years for experienced men to be attached to central fire stations to deal with water supply problems that may arise. In carrying out emergency repairs, not likely that run-lead-joint will be of much assistance. Each type of joint must be carefully considered. Every undertaking should

have available at least one portable chlorinating apparatus. As measure of guarantee of domestic supplies, tank wagons would be especially appreciated by public. Power valve-closing units would be particularly useful on large-size valves. Whatever form of organization for water service, it must not only ensure efficiency of its own service but at same time maintain closest cooperation with fire-fighting, de-contamination, and other local services, as well as work in a mutually beneficial manner with adjoining water suppliers. In dealing with repair work, conditions may be such that it be impossible to deal with all fractured mains at same time, and it is essential that those most likely to dislocate functions of other defense be tackled first.—*H. E. Babbitt.*

**Emergency Repair of Water Mains. Developments of Glenfield Systems.** ANON. Wtr. and Wtr. Eng. (Br.) **42:** 76 (Mar. '40). Exhaustive tests, both in Glenfield works and in field have proved system effective. Fitting, designed to be slipped on broken end of main, is secured by ring-nosed set screws and watertight joint is made with flexible sealing ring forced into contact with outside of main. "Connector" is a similar fitting, except it has standard end outlet on center line of main, a 6″ diam. outlet being common to all sizes connector. Another type of fitting, with branch for dewatering, is a coupling designed for attachment to broken end of a main. Experiences of modern warfare has shown that 2 principal types of bomb damage affecting mains are to be expected; damaged pipes may be buried under piles of debris, or craters may be formed interrupting main and exposing damaged ends. In first case, with Glenfield system, necessary only to dig down to main on each side of encumbered areas and cut it to permit couplings to be attached. Ends of the main may then be reunited by lengths of fire hose. Where craters have been formed emergency couplings can be connected quickly to broken ends. In connection with air raid precautions Glenfield multiple fire hydrant can be used. Consists of 6 standard fire hydrants mounted on a special casting with 3 branch pipes and breeches pieces. Can be connected to 8″ or larger pipe.—*H. E. Babbitt.*

**Inserting Valves in Water Mains Under Pressure.** ANON. Wtr. & Wtr. Eng. (Br.) **42:** 44 (Feb. '40). Under war conditions, particularly desirable to be able to insert valves in water mains under pressure to provide for alternative supplies and to permit interconnection of distribution systems of different authorities. Method followed is: (1) Pair of cast-iron collars is calked on pipe where valve is to be inserted. Fixed collars are provided with sliding collars, properly bored to fit valve flanges, and equipped with an adjustable gland to give watertight joint between sliding faces of two collars. (2) Pipe-cutting machine is assembled on pipe between collars. (3) Watertight casing to enclose collars and cutting machine assembled about pipe. Casing is provided with a chamber on top in which valve is housed. External drive to cutting machine is transmitted to it by means of shaft passing through gland in casing. (4) After pipe has been cut, cutting machine is withdrawn into chamber and valve is lowered, inside of chamber, into position between collars. (5) Sliding collars are moved forward on machined spigots of fixed collars, up to valve flanges. Bolts are set up through glands in casing. (6) Casing is removed and lead is run and calked between flanges of fixed and sliding collars. Apparatus is available for all mains up to 18″ in diam.—*H. E. Babbitt.*

**Water Supply for the Army.** ANON. Pub. Wks. **71:** 11: 28 (Nov. '40). Two problems connected with Army water supply deal with supply for fixed bases and supply for moving troops. General principles of municipal water supply practice applicable also to camps and fixed bases, with exception that population may be increased or decreased rapidly, and that useful life of plant is uncertain. With ample supply and no restrictions on use, Army records indicate use will run from 50 to 75 g.p.d. per man; lax water discipline may result in

consumption rate as high as 200 g.p.d. Peaks vary widely with greatest load probably in late afternoon. At one camp of 2,500, min. consumption per man at 11 A.M. was 1½ gal. per hr., with max. at 3:30 P.M. of 8½ gal. per hr.; daily consumption in this camp, 95 gal. per man. Believed mains and storage facilities should be designed to furnish 20 gal. per man per hr., perhaps more in hot climates. Usual Army practice to provide, per 1,000 men, 80 to 100 toilets, 150 lavatories, and 50 to 80 showers, practically all of which will be in use during hr. of max. consumption. Fire reserve should be in addition to peak load demands. Treatment should be in accordance with modern san. eng. practice. Quartermaster corps responsible for construction, maintenance and operation of water purification plants and distribution systems and for quantity and quality of water. Medical corps charged with making surveys, studies, examinations, and recommendations for continuous production of safe water. In temporary and semi-permanent installations, requirements are 25 35 g.p.d. per person; where water distributed from trucks and water carts, 5 gal. per day per person will be consumed and 10 gal. per day for each animal. In field camps and on march, requirements are 2 gal. per person per day and 10 gal. per animal. May be reduced to 2 qt. per day for men and 3 to 5 gal. for animals in time of combat. Improved portable filter unit has been developed in past 2 yr., weighs about 700 lb. and, when operated as filter, has capacity of 12 to 15 g.p.m., somewhat higher when not filtering. Used in '40 maneuvers and proved satisfactory. Provided with hypochlorinators for final treatment. Stress on mechanization and mobility has increased water supply and water haulage problems.—*Martin E. Flentje.*

**Water Supply and Sewage Disposal for Airports.** ANON. Pub. Wks. **71:** 11: 25 (Nov. '40). Cooperative construction of 4,000 local airports planned by gov't. agencies which furnish labor, municipalities and counties, which are preparing plans, supplying construction equipment and supervision of work. Needs for fire protection and demands of personnel, both permanent and transient, will determine design as regards quantity. Ave. airport has about 70 employees, to which must be added transients. Army air fields have larger personnel. Ave. of 13 Army fields personnel is 2,350, 700 being service and operating force. Of 13 fields, only 2 have fewer than 1,000 men and none over 3,180. 4 airports will have 175 planes based on them, and an ave. personnel of 1,800. Personnel divided into 3 categories: (1) those at field 24 hr. per day, for which 100 g.p.d. per capita should be furnished; (2) 8 to 10 hr. per day workers, 35 g.p.d.; (3) transients, 5 to 10 g.p.d., preferably 10. Mains should be designed to provide 2½ times estimated consumption and fire protection also provided for. Where hydraulic gasoline systems used, 50 to 100 g.p.m. required when in operation with min. residual pressure of 30 lb. Rate of use at Army air fields very high for certain periods of day. Standards for fire protection requirements for airports now undergoing revision. Design of sewage disposal plant important, should be amply large and allow for future expansion.—*Martin E. Flentje.*

## HEALTH AND HYGIENE

**The Work of the U. S. Public Health Service.** Supp. No. 152 to Pub. Health Rpts. ('40). (Revision of Reprint No. 1128.) U.S.P.H.S., established as Marine Hospital Service, July 16, '98, provided medical treatment and hospital care for sick and disabled seamen. Gradually, functions have been extended until now Service comprises 8 major divisions and spends about $14,000,000 annually. Primary objective is to reduce disease prevalence. To this end, Service functions to prevent introduction of disease from without; medically examines arriving aliens and prospective immigrants; prevents interstate spread

of disease and suppresses epidemics; cooperates with state and local health authorities; investigates diseases of man; supervises and controls biologic products; educates public and disseminates health information; maintains marine hospitals and relief stations for care and treatment of certain legal beneficiaries; confines and treats persons addicted to habit-forming narcotic drugs; studies mental diseases and drug addiction; and provides medical service in Federal prisons. Administrative divisions are: National Institute of Health, Marine Hospitals and Relief, Foreign and Insular Quarantine, Sanitary Reports and Statistics, Venereal Diseases, Mental Hygiene, and Personnel and Accounts. Among other functions, the Domestic Quarantine Div. exercises control over water supplies used for drinking and culinary purposes on railroads, vessels and other interstate carriers; cooperates with states in certification of interstate shellfish shippers. A wide variety of research activities include study of sewage and industrial waste disposal.—*Ralph E. Noble.*

**Water Supplies at Army Posts at the Time of Inauguration of the Sanitary Report.** GEORGE F. LULL. Military Surgeon **86:** 600 ('40). On Aug. 25, 1868, circular order was sent out from Surgeon General's Office, War Dept., directing senior medical officers on duty at a post to prepare sanitary reports twice a year of post bldgs., water supply, drainage, etc. On Dec. 5, 1870, results of the previous year's study published in volume known as S.G.O. Circular No. 4. During 1869, there were 150 posts and 25,927 soldiers scattered over U. S. Very little understood of water-borne diseases at this early date, and only effort made was to secure clear cool water especially during summer months. During 1869, diarrhea and dysentery varied from 197 per 1,000 in Dept. of Platte to 366 in the Dept. of South. Report prepared by medical officer showed that water was secured from many sources, raw river or creek water furnishing about ⅓ of supplies. Dual water supplies at many posts during certain times of year. Secondary supply used when original supply failed. During period that secondary supply was used, intestinal diseases were common. Fort Delaware was one of few stations where water treatment plant was installed. Description of method of purification and storage given. Civilian community water supplies were no better than army supplies, and intestinal diseases were prevalent, especially during the summer. Many years later, army stations were assured safe water supplies.—*P. H. E. A.*

**Ohio River Valley Water Sanitation Compact.** R. H. MARKWITH. Annual Report of Div. of San. Eng., Ohio Dept. of Health, 1939. p. 7. Compact adopted by 5 states—Ohio, N. Y., W. Va., Ill. and Ind. Actual adoption by Ohio and W. Va., however, must wait acceptance by other states listed in provision—Pa. in case of Ohio; and Pa. and Va. in case of W. Va. Delay foreseen in that no regular session of Pa. legislature is scheduled until '41.—*Ed.*

**Anti-Bathing Regulation Upheld by Master in Pleasant Pond Case.** ANON. N. H. Health News. **18:** 6 (July '40). Recent report of Master sustained N. H. Health Department in ruling prohibiting bathing in Pleasant Pond. Lake, a body of water of 450 acres, lies within towns of Northwood and Deerfield and constitutes water supply for Pembroke and adjacent communities. Because of increasing number of cottages and bathers, pond closed to bathing by order issued in '37. Validity of order disputed in court, plaintiffs (cottagers, towns of Northwood and Deerfield, and Power Co.) holding regulation to be unreasonable and unnecessary, that it entails deprivation of valuable property rights, that health of water consumers not endangered by such bathing and, if such were case, that Pembroke should be required to purify water. Health Dept. held pure municipal water supply to be of permanent importance, that bathing serves to pollute water and that purification of such water would not be simple and inexpensive matter. Assumed case will be taken to supreme court. Present

decision, however, of broad interest to water works and health authorities.—*Martin E. Flentje.*

**Healthier Health Meetings.** W. W. Bauer. Am. J. Pub. Health. **30:** 523 (May '40). Author pleads for thorough advance planning of health meetings on dates not conflicting with other important community affairs and planned with sufficient advance publicity. Speaker and organization should be on time; in extending invitation to speak, frankness advised with regard to ability or desire to pay or not to pay expenses and honorarium. Speakers should be sent to ready-made audiences as service and women's clubs, schools, etc. rather than to arrange for a public meeting with sole purpose of presenting a health talk. Speaker should have something to say and be able to say it in time allowed. A 2¼ hr., or shorter, suggested program listed, meeting author's suggestions for a health meeting that is neither overloaded nor abandoned to utter desolation.—*Martin E. Flentje.*

**The Significance of the Chemical Analysis in the Hygienic Judgment of Drinking Water.** Oskar Spitta. Gesundh. Ing. (Ger.) **63:** 455 (Sept. 7, '40). Historical refs. given that show that first only chem. anal. used in deciding water quality. With development of bacteriological methods, these became more and more important and some public health workers considered even that chem. anal. had no value in detn. of hygienic properties of water. Spitta, who, at 70, is still one of Germany's leading medical authorities on water, opposes this standpoint. Otherwise filtered and sterilized sewage could be used. Several references are cited showing that bacteriological and chem. results do not run parallel. Sanitary survey in field is necessary to interpret results. Chem. anal. gives soluble compounds that may come from surface contamination or from ground formations; bacteriological exam. shows filtering property of ground. Sanitary survey of local special conditions gives better standard for classification of permanent quality than one-sided bacteriological test. All three tests, chemical, bacteriological and sanitary have to cooperate whereby judgment has not only to decide on safe supply but also on one of wholesome origin.—*Max Suter.*

**Some Notes on Sanitary Land-Fills.** Lewis V. Carpenter and Lloyd R. Setter. Am. J. Pub. Health. **30:** 4: 385 ('40). Previously, land-fill disposal for garbage not studied sufficiently to draw conclusions as to economy and desirability. W.P.A. project under N. Y. City Health Dept. recently organized to investigate. At present, current observations, rather than final conclusions, available. From representative samples of mixed refuse, physical, chem., and bact. analyses made. Results tabulated according to depth and age of fill. In addition, gas and temp. studies made at various depths of fill. Last survey to date was on odors to determine limits of offense. Article gives complete procedure used and many tables of results found.—*P. H. E. A.*

**The Sanitary Fill as Used in Fresno.** Anon. Am. City. **55:** 2: 42 ('40). Author discusses procedure taken at Fresno, Calif. on disposal of garbage by sanitary-fill method. First sanitary fill made at city sewer farm and, after 2½ yr. operation, new site selected. Sanitary aspects involved in making fill include: (1) if garbage is not covered, fire will invariably break out and every endeavor should be made to prevent fires in fill because they interfere with anaerobic decomposition of garbage; and (2) precautions should be taken to prevent burrowing of rats and experience at Fresno is cited. Since no separation of garbage and refuse is required in Fresno, additional costs of collection are referred to. Complete cost data regarding sanitary-fill method for city also incorporated.—*P. H. E. A.*

**Garbage and Refuse Collection and Disposal Practices in Small Communities.** Pub. Wks. **71:** 2: 9 ('40). Article contains statistical data relative to garbage and refuse disposal at approx. 1,000 communities of less than 4,000

pop. Results of survey indicate that approx. 70% of communities have organized system of collecting refuse. Data further indicates no. of collections made weekly, method of collection, frequency of collection in winter and summer, charges assessed, and method of disposal. Of particular significance is fact that from information procured from communities questioned, 50.2% of communities dispose of their refuse on open dumps, 31.6% of it to hogs, 6.6% disposed on covered dumps, 6.2% disposed of by burial, 4.8% disposed of by incineration and 0.6% disposed of by sanitary fill method. Evident, in words of editors, that municipalities do not realize value of proper disposal of refuse and garbage. Information relative to geographical locations, methods of disposal, collection, and hauling given.—*P. H. E. A.*

**Court Decision on Public Health.** *"Sanitary tax" in connection with privies upheld.* (S. C. Supreme Ct.; *Town of Marion* v. *Baxley*; 5 S.E. 2d 573; decided 11-13-39.) Pub. Health Rpts. **55:** 785 (May 3, '40). Regulation of town board of health provided that every head of a family having a sanitary closet or privy should be liable for annual sanitary tax of $3. Regulation was adopted as ordinance, making violation of regulation punishable by fine or imprisonment. Resident, convicted for not paying tax, appealed to supreme court. Latter affirmed judgment appealed from, stating that: (1) Evident purpose of regulation was to impose an inspection or service charge upon designated class for service rendered and privilege permitted, while constitutional requirement that taxes levied must be uniform in respect to persons and property had no application. (2) Regulation was valid and board of health acted within scope of its delegated powers to protect and preserve public health. (3) In adopting ordinance, town acted within scope of its powers delegated by legislature, and enforcement of ordinance by fine or imprisonment was not violative of any constitutional provisions relied on by defendant.—*Ralph E. Noble.*

**Typhoid Carriers.** ANON. Lancet. (Br.) **2:** 1335 ('39). Fevers caused by *Eberthella typhi* not distinguished from those caused by *Salmonella paratyphi*. Spread of the disease through carriers is discussed. Statutory Rules and Orders No. 1004 ('27) gives power to prohibit employment of carriers in catering trades, but value of order is limited by number of complications. Carriers of typhoid and paratyphoid can both infect foodstuffs, and typhoid carriers can also infect water supply; it is uncertain whether paratyphoid can be spread by water. Major epidemics of typhoid are usually spread directly by water supply, or indirectly through water, e.g., in swimming pools. Control of carriers would not eliminate risk of infection of water supplies. Although Croydon epidemic might have been prevented by control of carriers, it could also have been prevented by other safeguards. Although segregation and treatment of typhoid carriers is not practicable, should be possible to compile register of them, as institutions at which cases of typhoid are treated usually determine whether a patient is a carrier before he is discharged. Such a record would not be complete, but it would make a certain amount of control possible. In '38, 1,322 cases of enteric fever and 163 deaths were notified. Fatality rate was higher than usual, and majority of cases were therefore probably due to *Eberthella*, as *Salmonella* is not frequently fatal.—*W. P. R.*

**The Efficacy of Immunization in an Acute Water-Borne Outbreak of Typhoid Fever.** C. D. BOWDOIN AND L. M. PETRIE. Med. Times. **68:** 53 ('40). In school of 323 pupils, water-borne outbreak of typhoid fever involved 27 persons, of whom 24 were aged 6–12 yr., attack rate in this age group being striking. Epidemic occurred in Oct. '37, and between '35 and time of outbreak 151 children had been immunized. Only 4 of these developed disease and all had been inoculated in Sept. '35, *i.e.* approximately 2 yr. before exposure, as were all except 7 of immunized groups. Finding that only such a small number

were infected is regarded by authors as demonstration of the efficacy of vaccination since attack rate was five times greater in non-immunized than immunized group.—*B. H.*

**Use of the Wilson and Blair Medium in Public Health Work.** CARRIEU, LISBONNE AND RAMBAULT. Rev. Hyg. Méd. Prévent. (Fr.) **61:** 571 ('39-40). Use of sodium and bismuth sulfites in nutritive media for isolation of *Eberthella typhi* and *Salmonella* was successful in most cases. Medium is useful in studying longevity of *E. typhi* in feces and sewage and in carrier detection.—*C. A.*

**Typhoid Outbreak Arouses Suspicion.** ANON. Ill. Health Messenger. **10:** 19: 111 ('38). Outbreaks of typhoid fever occurred at Bellewood and Danvers. Investigation showed that in former case, rain water, polluted by sewage, may have entered water main during road repairs when a sewer and a water main lying below it were open at same time; in latter case infection was due to pollution of two wells from a sewer. —*P. H. E. A.*

**53 Die in Typhoid Epidemic Caused by Water Contamination.** ANON. Hospital Mgmt. **48:** 27 (Nov. '39). Typhoid epidemic which resulted in illness of 500 and deaths of 53 persons occurred at Manteno State Hospital, Manteno, Ill. Source of epidemic traced to defective water supply, water for institution being obtained from 4 wells in limestone formation. Believed that water was contaminated by leaking sewer pipes at hospital but State Board of Health reported this water supply to be unsanitary as early as date of opening of hospital in '32. Supply is now being chlorinated, but water for drinking is being brought daily to hospital by truck from Kankakee, Ill., a distance of 13 mi.—*P. H. E. A.*

**An Outbreak of Typhoid Fever in the Municipality of Caloocan, Province of Rizal.** T. CORPUS AND J. T. SAN JUAN. Monthly Bul., Bur. of Health (Manila). **18:** 663 ('38). Between May and Oct., '38 there were 16 cases of typhoid fever in Caloocan, time distribution being May 1, June 3, July 1, Aug. 7, Sept. 3, October 1. Of the 16 cases, 9 were positive both by Widal test and blood culture and 7 were Widal positive only. Various sources of infection were studied and two were highly suspicious. Seven gave history of having taken oysters. Oyster beds are subject to pollution from two brooks into which town drainage goes, and from samples of oysters collected Sept. 24, typhoid bacillus was isolated. Some of inhabitants take water from open surface well which bacteriologically showed slight contamination (coliform in 1 ml.). Seven of cases were from near this surface well, but history of water being consumed by only one patient. Authors consider "that water is the most probable source of infection in this particular outbreak." ★[Evidence hardly supports this view; contamination was only slight for a surface well and distribution over six months is much more in favor of shellfish being responsible.]★—*B. H.*

**Outbreak of Typhoid Fever in Linares (Chile) in 1937.** J. DONOSO. Rev. Chilena de Hig. y Med. Prevent. (Chile) **1:** 37 ('37). For a long time Linares has been known as an endemic focus of typhoid fever, and analysis of previous outbreaks shows that 80% of cases occur in months Jan.–June. Disease when it becomes epidemic is water-borne and analyses of various samples demonstrated efficacy of methods of chlorination adopted, but—and this is important point—chlorination has not been continuous, sometimes several hours passing without chlorination, and water samples at these times contain dangerous amount of coliform. Also in urban area are many wells which are used as source of drinking water, and these show gross pollution. In outbreak referred to there were 40 cases, including 3 suspects; 18 were ascribed to overcrowding of persons with no hygienic sense, among whom were found carriers; 10 seemed to be due to drinking polluted water; and another 9 were probably ascribable to the same cause. Four deaths occurred. Remedy

is obvious, comprising better source of supply of drinking water, or effective continuous chlorination of present supply, closing down of wells in the town, improvement of sewage disposal, and vaccination of those exposed to risk. —*B. H.*

**A Small Domestic Epidemic of Paratyphoid-B-Schottmüller With Evidence That the Drinking Water Was the Probable Source of Infection.** L. HIPPE. Arch. Hyg. (Ger.) **121:** 56 ('38). At end of 19th century, over 70% of cases of typhoid were caused by infected water supplies, but, in more recent years, only small proportion of cases have been due to this cause. Contact has been chief way by which disease has been spread. Even in recent years, however, large and small water-borne epidemics have occurred. Evidence from literature of ability of pathogenic organisms to survive in water is discussed. Small epidemic of paratyphoid-B-Schottmüller in a house in a village in Schleswig-Holstein is described. In middle of Aug., infant developed the disease, and after 2 days was removed to a clinic. About 10 days later 3 further cases occurred among members of family who had not been in contact with original case. Paratyphoid-B bacteria of Schottmüller type were found in water from well serving house. In a later exam., these bacteria were absent. Seems probable that well was infected from a pit into which waste waters from house were thrown, and which had received water which had been used for washing sick infant. Cause of first case is unknown. Similar cases, which have been described by previous investigators, are summarized.—*W. P. R.*

**Court Decision on Public Health.** *Compensation for Typhoid Fever Under Workman's Compensation Act Denied. Hoffman et ux.* v. *Consumers Water Co. et al.* (Idaho Sup. Ct.; 99, P2d, 919; decided Feb. 23, '40). Pub. Health Repts. **55:** 1179 (June 28, '40). Proceeding under Idaho Workmen's Compensation Act to recover compensation for death of employee from typhoid fever. Deceased had been employed in cleaning an open irrigation ditch containing muddy pools of waste water harboring dead animals and waste matter. Attending physicians held opinion that infection came from ditch where deceased worked. Testimony not contradicted. Question presented to supreme court was whether typhoid fever in case was accidental injury incurred in course of, and arising out of, his employment. Supreme court sustained appellate court and decided, "no evidence whatever the deceased was conscious of mishap, hazard, fortuitous occurrence, or misadventure from or by reason of which he sustained an injury"; nor "evidence of an accident resulting in an injury to deceased which would bring the case at bar within" certain cited cases.—*Ralph E. Noble.*

**Fluorides in Food and Drinking Water.** F. J. MCCLURE. Nat. Inst. Health, Bul. No. 172 ('39). Following essential information presented in author's approach, materially helps to better understanding of general fluorosis problem and conclusions based on his expts. Human fluorosis, or mottled enamel, is restricted to areas where domestic water contains toxic quantities of fluorides (F's). Limited to endemic hypoplasia of permanent teeth. In severe cases, brown stain is a secondary complication. Importance of food as F vector is largely unknown. In general, F-containing foods do not constitute an appreciable portion of infants and childrens diet but, when they do, mottled enamel may occur only if certain min. quantities of F are ingested regularly during development of crowns, except 3rd molars, from birth to 8 yr. Ultimate effects of F-containing food and water depend on total of both consumed regularly. Possible that natural F-content of food may not be as toxic as inorganic F compounds *per se* or in water. Thus, mode of ingestion might differentiate between effects of food F's or water F's. Earlier expts. with rats indicated 500 p.p.m. NaF in drinking water produced relatively greater toxicity than same amt. in diet. Differences in total F ingested, relative degree and rates of absorbtion, and other factors, however, condition these results.

Other workers using different methods found little difference attributable to food versus water ingestion. Bones and teeth are main F depositories. Author's experimental plan was well organized and controlled. Careful consideration was given to the chemistry of F determination. Conclusions follow: With growing young rats as experimental animal, chronic fluorosis was studied particularly from standpoint of severity of effects produced by 22.6, 45.2, and 90.4 p.p.m. equivalent of F ingested in food, compared with similar quantities ingested in drinking water. So far as may be shown by (1) ave. daily body gain, (2) ash and F content of bones and teeth, (3) appearance of teeth, and (4) total F retained in final body wt., no noticeable differences could be attributed to either of two mediums of F ingestion. Acute toxic effects of approx. 180 p.p.m. F as NaF in drinking water was noted, confirming previous observations. These, however, are not to be confused with conditions and effects accompanying F consumption in drinking waters of mottled enamel areas. Ave. total retention of F from NaF may equal 30 to 40% of intake, at concentrations studied. By far, greater portion of this F is deposited in bones and teeth. Degree of tooth hypoplasia in rat is directly correlated with quantity of F present. Has been suggested that F content of bones and teeth of expt. animals may be developed into biologic test for effective F's in foods as well as a means of indicating pathologic condition in affected tissue. In young rats: (1) there is a suggested stimulating effect of 22.6 p.p.m. F in diet, on appetite and perhaps on daily body gain; (2) stimulating effects of F's on water-drinking again noted; (3) NaF equivalent to 45 p.p.m. or more F in food and water indicated reduced rate of gain; (4) increased ash content of bones occurred from NaF ingestion equaling 22.6 to 600 p.p.m. F. In rat, 0.03-0.04% F represents approx. max. that may be present in whole tooth and tooth enamel still retain normal macroscopic appearance. Quantities of F in large no. of foods, as reported in literature, are summarized. Although extremely difficult to judge actual conditions of ingestion, and while positive statements are not warranted, would appear that accumulation of F in bones of adults living in areas where mottled enamel is endemic may be expected. Accumulation of abnormal and perhaps pathologic quantities of F in bones of adults in such areas appears probable. Future study of chronic fluorosis requires placement of emphasis on localization of F, particularly in bones and teeth, relating directly to cumulative effects of F ingested. More information is needed relative to quantities of F's in foods, toxicity of natural food F's as compared with pure inorganic F's, associative factors involved in F absorption and metabolism. Direct information is needed from epidemiological biochemical studies relative to involvement particularly of bone tissue among human populations of mottled enamel areas. 4 graphs, 6 tables, 27 photographs and 89 refs.—*Ralph E. Noble.*

**Occurrence, Pathological Aspects, and Treatment of Fluoride Waters.** M. S. NICHOLS. Am. J. Pub. Health. **29:** 991 ('39). To prevent mottled-enamel condition in teeth it is necessary to provide drinking water with fluoride content of less than 1 p.p.m. A table shows association of mottled enamel with high fluoride content; with 4.4 p.p.m., 97.6% of children, aged 9 to 11, were affected. Waters containing excessive amounts of fluoride are widespread throughout U. S., and evidence is accumulating that this state of affairs is world-wide. In general, fluoride waters are derived from wells, origin of fluorine being igneous rocks and, to lesser extent, sedimentary formation where vertical seams of fluorspar are found deposited by thermal waters laden with fluorides and fluosilicates arising from igneous formations, and spreading laterally through faults and fissures of sedimentary layers. Fluorapatite, however, is common constituent of all classes of rock and is used as a fertilizer, a supplementary mineral in stock feeding, and in baking powders. With regard to the pathological aspects of fluorides, De Eds and McClure have made exhaustive reviews

of subject. As little as 4.0 grams of fluoride as a single dose by mouth has proved fatal. Chronic poisoning by fluorides is more difficult to assess; 1.5 gram of fluoride is found to be upper limit of tolerance for cows. No definite evidence that fluorine is essential for bodily function, but much evidence that it is a poison. Dean has recently found that dental caries attack-rate in children using high fluoride waters is only ½ of what it is in those drinking waters low in fluorides. Treatment of high-fluoride-content waters is still a matter of experiment, but, recently, Behrman and Gustafson have improved method of Adler, Klein and Lindsay. This consists of preparation of layer of "Defluorite" (hydroxy apatite, $3Ca_3P_2O_8Ca(OH)_2$ and tricalcium phosphate, $Ca_3P_2O_8.H_2O$ from commercial phosphoric acid of high purity and milk of lime). Water is passed through layer and regeneration is carried out with 1% caustic soda followed by carbon dioxide for neutralization of residual alkali. Such a system can be used indefinitely, and is in part covered by a patent application. Individual household units have been proposed, but would require careful control or people would be laboring under a false sense of security.—*B. H.*

**Effect of Fluorine on Dental Enamel, and the Fluorine Content of Some Hungarian Potable Waters.** JANOS STRAUB. Orvosi Hetilap. (Hung.) **84:** 121 ('40). Of 80 samples of well water examined, 2 samples contained 858 and 809, and 11 more than 500γ fluorine per liter. Water of Budapest aqueduct contained 186γ fluorine per liter. Relation between fluorine content of waters and dental disorders cannot yet be proved in Hungary.—*C. A.*

**Occurrence of Fluorine in the Drinking Water of New Mexico and the Menace of Fluorine to Health.** JOHN D. CLARK AND EDWARD H. MANN. Univ. New Mex. Bul., Chem Ser. 2, No. 5. ('38). Of 157 different samples examd. for F, 35 contained more than 0.9 p.p.m., which is considered danger point. 22 addnl. communities had water supplies contg. more than 0.8 p.p.m. of F. Map is included giving location of high F water supplies. While mottled enamel in teeth is commonly encountered in N. M., believed that no bone injury results unless water contains at least 6 p.p.m. Physiol. effects of F on animal system are reviewed. Methods suitable for F removal from domestic water supplies are being studied. Addn. of bone ash followed by boiling the water and allowing ppt. to settle removes much of F and may be a useful method for home use.—*C. A.*

**Dental Fluorosis in Mexico.** LUIS MAZZOTTI AND M. GONZALEZ. Riv. del Inst. de Salub. y Enferm. Trop. (Mex.) **105:** 1 (Nov. '39). Report of preliminary survey undertaken in cooperation with physicians of Pub. Health Dept. to determine extent of fluoride distribution in water supplies and to detect presence of dental fluorosis in Mex. Of 107 samples analyzed, only 5 were fluoride-free. Concentrations upwards of 0.9 p.p.m. were found in at least one sample from each of 11 states and 2 territories of 22 political subdivisions included in survey. In general, no modification in dental enamel was noted in areas where fluoride free water was used. Where concentration ranged from 0.1 to 0.4 p.p.m., difficult to detect signs of mottled enamel. Where concentration ranged from 0.5 to 0.9 p.p.m., a few cases of affected enamel were noted, especially as concentration approached 1 p.p.m. Waters with fluoride content exceeding 1 p.p.m. associated practically always with mottled enamel production of varying degrees of severity. Highest fluoride concentration found was 17.5 p.p.m., there being waters from 4 other localities which contained from 4 to 7.5 p.p.m. Results of survey seem to indicate that dental fluorosis in Mex. resembles more closely "mottled enamel" of U. S. than "darmous" of northern Africa. Since there are still many places where no survey made, presence of dental fluorosis in Mexico is probably much more extensive than progress report indicates.—*J. M. Sanchis.*

**Endemic Fluorosis in South India.** C. G. PANDIT. Bull. Mens. Office

Intern. Hyg. Publ. (Fr.) **32:** 450 ('40). High incidence of spotted enamel on teeth of children and of spinal rigidity in adults was correlated with presence of fluorides in unusually large proportions in drinking water.—*C. A.*

**Gastro-Intestinal Derangement During Droughts.** GEO. G. SCHAUT. Am. J. Pharm. **112:** 183 ('40). Exptl. evidence is presented that continuous administration of an alkali cyanide in sublethal concn., through medium of their drinking water during several days, caused a mild form of gastro-intestinal derangement in rats and dogs. Alkali cyanide in natural water was partially destroyed by boiling and completely eliminated by boiling when in distd. water. Water contg. Cl is de-chlorinated immediately in oral cavity of human subjects. 22 refs.—*C. A.*

**Court Decision on Public Health.** *Pollution of City Water Supply.* (Miss. Supreme Ct., Div. A: *Carey-Reed Co., Inc.* v. *Farmer*; 192 So. 48; decided Nov. 20, '39.) Pub. Health Rpts. **55:** 849 (May 10, '40). Action was brought to recover damages for injuries alleged to have resulted from drinking water from city supply which plaintiff claimed had become polluted through negligence of company engaged in laying concrete paving on highway between 2 municipalities 3 miles apart. Without knowledge or consent from Cleveland authorities, company made cross-connection between sewage polluted bayou, at Boyle, and municipal supply. Pressure on bayou line was 125 lb. as against 40 lb. in city supply. On certain date, pump at Boyle ran all night. Out of 366 persons in area about cross-connection, 157 were sick from acute gastro-enteritis caused by polluted water. Court found in favor of plaintiff.—*Ralph E. Noble.*

**Dysentery Outbreak Due to Polluted Water.** ANON. N. H. Health News. **16:** 14 ('38). During fortnight in May 1938, 71 cases of gastro-intestinal disease occurred in 2 villages which had a common water supply, obtained from a creek. Water is chlorinated but not filtered, and before outbreak, chlorination was interrupted on at least one occasion. At same time heavy rain increased pollution at source. Outbreak emphasizes fact that chlorination alone is not sufficient to render a contaminated water safe. ★[This remark does not seem to be justified as the chlorination had not been continuous.]★—*W. P. R.*

**A Third Outbreak of Staphylococcal Food Poisoning in Hamilton, Ontario.** J. ROBERTS. *and* **Laboratory Findings.** R. J. WILSON. Canadian Pub. Health J. **30:** 590 ('39). About 140 cases in 63 families. Symptoms, diarrhea and vomiting; no deaths. Incubation period 1 to 6 hr. Duration of illness 2-15 hr. Majority of cases attended local church dinner and of 102 persons who ate affected food 96 became ill. Epidemiological evidence showed that coconut pie, served as dessert, was responsible. Similar pies distributed elsewhere from same bakery accounted for rest of cases. Pies were made in late afternoon of Mar. 22, '39, placed on racks, many pies still warm, in what was considered a cool room, until delivered about 24 hr. afterwards to the church and eaten the evening of March 23. Sanitary conditions at bakery were satisfactory. Cultures of *Staphylococcus aureus* isolated from cream pie and from a number of sufferers. Cultures of 5 out of 8 bakery employees showed this staphylococcus in nose and throat. Of interest that similar outbreaks in Mar. and May '38 originated from same bakery. Owner harbored same strain of staphylococcus in his nose and throat from Mar. '38 to April '39 in spite of medical treatment. He apparently infected other members of staff. Employee in charge of fillings showed same staphylococcus at time of outbreak and also 4 months afterwards. —*B. H.*

**The Food Handler as a Transmitter of Amebiasis.** A. W. SCHOENLEBER. Am. J. Trop. Med. **20:** 99 ('40). Author criticizes severely experimental work of Sapero and Johnson and their conclusion that food handlers are of minor importance in amebiasis transmission. At Aruba, an island off Venezuela, are about 1,500 Americans living under

conditions which preclude transmission of infection from water, flies, sewage, or soil contamination of food. They arrive direct from America so their average amebic infection rate is not more than 10%. When population was investigated after several days' residence, results showed that 33% of native population, including cooks and servants, were infected with *Entamoeba histolytica* and 25.57% of American population. Active measures for control of dissemination of infection by food handlers were put into effect, but no other control measures. Percentage infected was reduced from 25.57 in '35 to 12.69, 5.70 and 1.92 in '36, '37 and '38 respectively. Author considers that food handlers are an important means of transmission.—*B. H.*

**Poliomyelitis Virus in Sewer Water.** ANON. J. Am. Med. Assn. **114:** 2232 (June 1, '40). 70 cases of poliomyelitis were reported in epidemic in Stockholm during summer months of '39. Dr. Kling, director of serotherapeutic institute of Stockholm, inoculated a number of monkeys with samples of contents of city's sewer waters. Repeated inoculation tests on animals elicited positive reactions. Thus confirmed observations of Paul, Trask and Culotta, made at Charleston, S. C., that viruses eliminated in fecal discharges may survive for certain length of time in sewer waters.—*J. H. O'Neill.*

**Tularaemia Infection Found in Streams.** ANON. Pub. Health Rpts. **55:** 227 (Feb. 9, '40). Rocky Mt. Lab. of U.S.P.H.S. reports 3 Montana streams contaminated with *Bacterium tularense*. 5 and 10-ml. portions of water, and a small amount of pool mud, infected guinea pigs.—*Ralph E. Noble.*

**The Solution of Lead In Drinking Water.** H. HAUPT. Gesundh. Ing. (Ger.) **62:** 161 ('39). Discusses mechanism whereby waters containing dissolved oxygen and carbon dioxide dissolve lead from lead pipes, and suggests means to reduce danger of lead poisoning. In older houses with lead pipes, change in quality of water supply can cause solution of protective coating which usually forms on pipe, so leading to new outbreaks of poisoning. Distilled water, saturated with air, will dissolve 85 mg. of lead per liter or, if saturated with carbon dioxide, it will dissolve 16 mg. per liter. Water of this kind, collected in rain-water cisterns, should never flow through lead pipes. If oxygen alone is present, lead hydroxide is formed; with small amounts of carbon dioxide, a basic lead carbonate forms (solubility 0.4 mg. per liter) which, with calcium carbonate and iron oxide, forms a dense protective coating, preventing further solution of lead; as concentration of free carbon dioxide rises, lead carbonate (solubility 1.1 mg. per liter) or bicarbonate (easily soluble) are formed. Hard waters with a hardness greater than 7° (German) form coatings within a few weeks. Soft waters containing iron seldom form protective coatings; iron oxide is apparently unable to adhere to smooth walls of lead pipe. Until protective coating has formed on new pipes, water which has been standing in pipes should be run to waste. Plants for removal of acidity and for adding lime to soft waters should be inspected regularly to prevent the entry of untreated water in the distribution system where previously formed coatings would be dissolved. In lead pipes treated with sulfur compounds, lead sulfide may be oxidized to soluble lead sulfate and the process therefore does not give adequate protection. Tin-lined lead pipes are liable to electrolytic corrosion which uncovers the lead.—*W. P. R.*

**Basophilia From Drinking Water Containing Lead.** A. WINKLER AND H. WIESBROCK. Z. Hyg. Infektionskrankh. (Ger.) **121:** 433 ('39). In a district where drinking water contained up to 1.1 p.p.m. lead, 24.3% of 375 children examined showed over 500 basophil erythrocytes per million erythrocytes in the blood. Max. was 2,300 per million and ave. for all subjects 230. In another district where water supply was satisfactory 318 children were examined and showed ave. of 50 basophil cells per million, with values of 300–500 in 2.8% of cases, and no values above 500.—*C. A.*

**Water Poisoning in Man and Animal, Together With a Discussion on Urinary Calculi.** Douw G. Steyn and Norman Reinach. Onderstepoort J. Vet. Sci. Animal Ind. (Pretoria, S. Afr.) **12:** 167 ('39). From results of expts. conducted by various workers, and from analysis of specimens of water and observations made by authors, it appears that extreme limit of total salt content of drinking water, especially for human beings, should be 570 p.p.m. Limit depends to large extent on nature and quantity of respective salts present; and suitability of each mineralized water for drinking purposes should be considered on its own merits (analysis). Most poisonous ingredients of mineralized waters are nitrates, sulfides and fluorides, while Mg, Ca and sulfates tend to upset normal processes of mineral metabolism. Carbonates and bicarbonates of Na and K render gastro-intestinal juices abnormally alkaline, disturb digestive processes and cause deposition of urates in kidneys. Methods of preventing poisoning by water are discussed.—*C. A.*

**Sulfur Dioxide Poisoning as a Cause of Asthma.** Alfred Romanoff. J. Allergy. **10:** 166 ('39). Three cases of asthma in individuals exposed to sulfur dioxide fumes from leaking refrigerators indicate industrial hazard of sulfur dioxide, particularly for those with allergic background.—*C. A.*

## BACTERIOLOGY

**The Scientific Foundations of the Bacteriological Tests of Water.** C. R. Baier. Gas u. Wasser. (Ger.) **83:** 329 (June 13, '40). Hygienic control of drinking water is generally done by 2 bacteriological tests—total count and determination of presence of coliform bacteria. Certain chemical tests ($NH_3$, $NO_2$, $O_2$, $SH_2$) also give some insight on bacteriological activity. So-called total count on gelatin or agar gives incorrect count of all bacteria. Autotrophic bacteria and those liking only small amounts of organic matter do not grow as well as certain symbiotic and parasitic organisms. Some bacteria grow so slowly that they do not show up during practical time of incubation. Mentioned as other factors that effect a low count are: oxygen tension, temperature and reaction of media, clumping of organisms, overgrowing of certain species. Irregular results are also due to non-uniform distribution of bacteria in water. Nevertheless, plate method gives dependable picture of aerobic saprophites. Incubation periods of 6 to 8 days give higher counts than only a 48-hour period, although liquefaction of gelatin is an obstacle. Agar gives lower counts than gelatin, but shows organisms of fecal origin more clearly. Direct microscopical count gives all organisms, even dead ones. Coliform test determines only certain physiological group. German methods try not only to show presence of coliform organisms but also to get approximate count of them, either by using different dilutions in liquid media or by direct plating on Endo agar. As incubation at 37°C. also gives coliform organisms from cold-blooded animals, method by Eijkman (incubation at 46°C.) is considered of more hygienic value, being specific for thermophilic gas formers. Many additional tests are possible and should be used to determine bacteriology and bacteriological history of a water. Variations in concentration of media often allows detection of physiological groups of different origin. More specialization is obtained by tests for certain definite organisms, as some pathogens, soil forms, serratia prodigiosum. Often important is the determination of bacterial activity by measuring chemical changes caused by bacteria, either by putting isolated bacteria in favorable media or by more natural method of putting chemical food to large water sample. Method also allows study of certain influences, as temperature, $O_2$ content and effect of poisons. Chemical analysis can give chemical activity directly in water course. Interpretation of results from bacteriological standpoint, however, requires care in differentiating biological and purely chemical causes of changes observed.—*Max Suter.*

**The Bacteriological Examination of Water Supplies.** *Ministry of Health.* ANON. Bul. Hyg. (Br.) **14:** 8: 630 (Aug. '39). Owing to variation of bacterial flora of different waters and varying local conditions of supplies, no hard-and-fast rules can be laid down for examination and interpretation of results. In water, as in milk control, it is far more important to examine numerous samples by simple test than occasional samples by more complicated test or series of tests. Three types of bacteriological examination are suggested; namely, simple, partial, and full. Standards are suggested to which it may be expected that public supplies should conform and these may vary with types and treatment of water. Standards are not to be interpreted too rigidly but serious discrepancies should be investigated promptly. Experience alone will dictate to bacteriologist whether in any given supply, departure from usual quality is significant or not. Standards are laid down on basis of presumptive coliform count in 100 ml. piped supplies: presumptive coliform count per 100 ml. —Class 1, highly satisfactory, less than 1; Class 2, satisfactory, 1–2; Class 3, suspicious, 3–10; Class 4, unsatisfactory, greater than 10.—*P. H. E. A.*

**Freshwater Biological Association of the British Empire. Eighth Annual Report.** (Year ending Mar. 31, '40.) Aims and progress of research carried out by assn. described. Results obtained from prelimin. studies on life history of salmon and on brown trout, rainbow trout, perch, minnows, and eels summarized. Prelimin. expts. on growth of trout indicate that treatment of tarns and lochs with fertilizers might be of practical value in improving fisheries. Comprehensive report prepared on previous literature on direct and indirect effects of fertilizers on fish. Studies on algae described. Bacteriological investigations for Dept. of Scientific and Industrial Research have concerned mainly fluctuations in numbers of bacteria normally existing in natural waters, and causes of fluctuations. Estimations of coliform bacteria showed no correlation between numbers of these bacteria and of bacteria as estimated from plate counts in lakes studied. In summer, coliform bacteria are most numerous in epilimnion; in winter, vertical distribution is more even. This is due to seasonal differences in circulation of water. In Trout Beck, flowing into Windermere, numbers of total and of coliform bacteria increased with increased flow, which washed into stream accumulated organic materials, detritus, etc. In Rothay River, receiving effluent from Ambleside sewage works, high river flow dilutes effluent, and no. of bacteria are decreased. Similar reactions found in other rivers. Lab. expts. on causes of fluctuations in numbers of bacteria indicate that insufficient food supply in lake water is not a factor limiting multiplication of bacteria. Studies of individual groups of bacteria are in progress. Future investigations will concern effects of rain water, organic matter, and carbon-nitrogen ratio on multiplication of bacteria, and functions of the various groups of bacteria. For work on lake deposits, new type of mud-sampler constructed. Surveys made of several lakes with echo-sounding machines have been analyzed and plankton in mud have been identified and counted. Chem. and hydrographic studies carried out along with bact. and algological investigations and have included study of processes of oxidation and reduction in water and mud. Investigations of coarse fish include study of feeding habits and growth rate of as many fish as possible in several different localities. Summary of results in tabular form. Finally, a brief report of work of Univ. College, Southampton, Branch for Southern Rivers.—*W. P. R.*

**Some Functions of Bacteria in the Purification of Polluted Water.** C. T. BUTTERFIELD. J. Bact. **39:** 527 (May '40). Discussion concerns bacterial purif. of water under aerobic conditions at 20°C. when there are involved: (1) dilute media or moderately polluted water having a 5-day B.O.D. of 6.0 p.p.m. or less; and (2) concentrated media or grossly polluted water having a 5-day B.O.D. of 100.0 p.p.m. or more. Under first condition, D.O. was not

utilized in absence of bacterial activity. Bacteria increased rapidly to a max. population accompanied by fast utilization of $O_2$, which then practically ceased. Protozoa failed to grow or use $O_2$ without bacteria, but would do both in concentrated media, or with bacteria in dilute media. Thus bacteria apparently act as "concentrators" of protozoan food in latter. With bacteria in pure culture, total $O_2$ used represents only half, or less, of the 5-day B.O.D. requirement; but with increasing complexes of bacterial and protozoan populations, max. bacterial levels are not reached, and oxidation approaches completion. Under second condition, with mixed flora and fauna, resultant predominating bacterial species were invariably types which grew in colonial form, or flocs, in the aerated liquid medium. Frequent replenishment of organic matter (bacterial food) increased floc enormously. Bacteria-producing flocs or colonies, in concentrated aerated liquid media, invariably failed to grow in dilute media, indicating insufficient nutriment to satisfy adsorptive method of feeding. *Colpidium* and a floc-producing bacterium did not accelerate or increase amount of purif. in first 3 to 5 hr. aeration, indicating growth stimulating agencies such as protozoa are not involved under these conditions. Continuous and successful purif. requires removal of excess bacterial floc and constitutes a major problem. In general, then, bacteria-producing cells in a dispersed condition only, must compete for food individually. Resulting zones of adsorptive influence soon develop and interfere with degree of food adsorbed. Those producing a floc or colony in a liquid medium must be kept in suspension to assure max. purif. and require an extensive adsorbing agent. Latter is represented by the bacterial cell capsules apparently blended together forming a continuous mass in the floc which has a high adsorptive capacity. Functions of bacteria in purif. of polluted waters are same regardless of degree of pollution. Feeding habits involved, however, determine extent to which purif. proceeds without natural or artificial assistance.—*Ralph E. Noble.*

**The Implications of Variability on the Species Concept in Bacteriology.** R. E. BUCHANAN. (*Published in abstract only.*) J. Bact. **40:** 153 (July '40). Among bacteria each cell is to be regarded as an individual. Bacteria respond readily to changes in environment; in many cases they also show marked genic instability. Care must be exercised in distinguishing genetically expressed from environmentally impressed variations among bacteria. There is no adequate proof either of sexuality or of its absence among bacteria. Evidence of presence or absence of a bacterial nucleus is still inconclusive, but that they possess functional equivalent of one is highly probable. Tendency to confuse genic variations in bacteria with some type of alternating mechanism corresponding to alternation of generations in higher plants. Generally accepted hypotheses used by geneticists to account for variation in higher forms, particularly fungi, should be tested out as probably adequate for bacteria. Genic mutations may well account for most bacterial variation. Equations are developed to illustrate what happens in a culture of bacteria when a mutation is being produced regularly in a definite fraction of normal cells.—*Ralph E. Noble.*

**Anomalous Results in the Bacteriological Examination of Water Supplies.** EDWARD HUGHES AND ERIC WORDLEY. Wtr. and Wtr. Eng. (Br.) **42:** 68 (Mar. '40) EDWARD HUGHES: Once a month strictly comparable samples were taken from area of supply of each res. and sent, one to Plymouth and one to an independent lab. Unfortunately, results from 2 labs. frequently disagreed. Certain unavoidable sources of error in any bacteriological exam., but one wonders whether there may also be avoidable errors in bacteriological practice. After some hesitation, was decided to send 3 comparable samples to 3 different labs. all of first rate standing, and in another case 4 comparable samples were sent to 3 different labs. Results were startling. One lab. reported all samples as highly satisfactory, another as 50% satisfactory, and third as all extremely unsatis-

factory. Expt. was repeated shortly afterwards with same results. In another test 8 samples of same tap water were sent to 2 labs., 4 to each. Results were strictly comparable and satisfactory. On repetition of test results varied from suggesting highly satisfactory water to seriously contaminated one. Bacteriologists will say that there is an error in our method of taking samples. "Candidly I feel that error (if any) is at one of the 3 labs. and I look forward to bacteriologists' explanation. Next point to consider is appropriate action to be taken. What is value of bacteriological exam. if results can be so divergent?" Experience suggests that peace of mind may well depend more on particular lab. which examines water samples than on danger of pollution itself. ERIC WORDLEY: Roughly speaking water examined locally immediately on receipt appeared to be excellent while those samples that had traveled were very much worse. Bearing in mind that Plymouth water supply is chlorinated moorland water, differences in bacteriological results mean either gross errors in technique, in sampling, or an enormous growth in transit due to unsatisfactory cooling. Many possible sources of error in lab. work. Real check on any lab's. work is that its results are consistent with probabilities. Apart from errors of technique is effect of lapse of time in transit. Inspectors could easily contaminate water during sampling. Last is possibility of dirty taps, from which samples were drawn. Presumptive tests for *Esch. coli* are only indicator of possible contamination and do not differentiate between *Esch. coli* and intermediates. So far as Plymouth was concerned, practically all positive presumptive *Esch. coli* tests were due to intermediates. Ideal plan is for exam. of waters to be carried out if possible by a nearby lab. so that constant personal collaboration between medical officer of health, water engineer, and director of lab. is possible.—*H. E. Babbitt.*

**Ways for the Natural Sanitation of Water. The Role of Protozoa and Bacteriophages.** (O.) Spitta. Gas u. Wasser. (Ger.) **83:** 293 (June 22, '40). No sharp limit between water and sewage. Water has to be free of germs causing diseases and of certain toxic substances. Besides these requirements of hygiene, esthetic condition also has to be fulfilled. Great number of bacteria present in polluted water is very important for its natural sanitation. Self purification should be encouraged, if need be by arranging favorable conditions to increase oxygen content, which is needed for activity of non-pathogen aerobic bacteria doing the work. Resulting self-purified water contains few bacteria owing to lack of food. Indications exist that pathogen bacteria disappear first. Besides activity of bacteria, dilution, sedimentation and effect of light also play part in process. Still another important factor lies in activity of protozoa. Studies show that these feed, mainly, on organisms foreign to natural water, such as pathogen and coliform bacteria. Chem. changes of dissolved organic matter are produced by aerobic bacteria; protozoa regulate bacterial content. Natural self purification can be disturbed by lack of oxygen and by secondary growth owing to a too rich food content. In both cases, reduction of food inflow can bring improvement. Self purification of surface water not sufficient to produce drinking water. Nature provides additional purification through filtering in ground water. Another help in eliminating bacteria is provided by nature in presence of bacteriophage. Simple test for bacteriophage described. Its value as measure of pollution is still doubtful. Nevertheless, bacteriophage helps to explain many obscure processes in self purification of water, such as difficulty of finding typhoid organisms in water.—*Max Suter.*

**Most Probable Numbers for Evaluation of Coli Aerogenes Tests by Fermentation Tube Method.** J. K. HOSKINS. *Reprint 1621.* 1940 revision of Pub. Health. Rpts. **49:** 393 (Mar. 23, '34). (*See* Jour. A. W. W. A. **27:** 407 ('35).) Author presents formula for calculating most probable number (M.P.N.) of coliform organisms in water from any possible combination of analytical results, regardless of number of tubes or dilutions,

and whether or not in geometric series (10.0, 1.0, 0.1 ml., etc.) when: (1) all tubes are inoculated with same amount of sample, e.g., 10.0 ml.; (2) equal or random number of test portions are inoculated in geometric relation to each other, e.g., 5-10.0, 5-1.0, 5 0.1 ml.; or 5-10.0, 1-1.0, 1-0.1 ml.; or 5-1.0, 7-0.1, 10-0.01, 8-0.001 ml.; and (3) random number of test portions are inoculated in various dilutions, not in geometric relation, e.g., 7-10.0, 12 5.0, 7-2.0, 8-1.0 ml. One table gives values for deriving M.P.N. from given combination, by formula. Eight tables list practically every combination encountered in above three cases, with corresponding M.P.N. equivalent, thus making it unnecessary to use formula for those combinations. By this method, lowest values are definitely limited by size of *largest* test portion; less limited by number of portions at this dilution; and not at all by increasing portions smaller than largest one. Thus, lowest M.P.N. obtainable from 1-10.0 ml. positive tube ranges from 23 per 100.0 ml. in the series 1-10.0, 1-1.0, 1-0.1 ml. to 2 per 100.0 ml. when it is 5-10.0, 5 1.0, 5 0.1 ml. Principle is particularly applicable to evaluation of drinking water required to meet Treasury Standard of 1.05 coliform organisms per 100.0 ml. No reasonable number of 10.0 ml. portions tested in accordance with 1925 Treasury Standard will measure content much below 2 per 100.0 ml. That measuring stick is too coarse. Author's method readily overcomes difficulty by testing portions larger than 10.0 ml. For example, 5-100.0 ml. give M.P.N.'s ranging from 0.22 to 1.6 per 100.0 ml., while 5-50.0 ml. give 0.44 to 3.2 per 100.0 ml. Suggested combinations of portions and dilutions are given for raw, applied, filtered and chlorinated waters. ★[Article makes available 3,813 possible M.P.N. values obtainable when planting 1 to 5 portions at each point in the series 10.0, 1.0, 0.1 ml.; 50.0, 10.0, 1.0 ml.; and 100.0, 50.0, 10.0 ml. This is a marked increase over 236 M.P.N. values cited in 8th ed. ('36) *Standard Methods* from same author's work in '34. At that time principle had not been expanded to include large portions of 50.0 and 100.0 ml.]★—*Ralph E. Noble.*

**Evaluation of the Errors Involved in Estimating Bacterial Numbers by the Plating Method.** MARSHALL W. JENNISON AND GEORGE P. WADSWORTH. J. Bact. **39:** 389 (Apr. '40). *Dilution error, i.e.*, errors of pipetting involved in reaching given dilution, and *distribution error, i.e.*, variation in number of colonies, due to sampling, between replicate plates of given dilution, are chief sources of variation accounting for *total error* involved in estimating bacterial numbers by plating method. An expression is derived for evaluating *dilution error*, deviations in pipettes and dilution blanks being known. Convenient table of dilution errors, as percentage standard deviations, is given, covering range of dilutions ordinarily employed, for all combinations of dilution blanks and for various deviations in pipettes and blanks. Usual method for estimating *distribution error* is indicated, and a formula for calculating *total error* from *distribution error* and *dilution error*.—*Ralph E. Noble.*

**The Enumeration of Bacteria by the Microscopic Method.** JOHN H. HANKS AND DAVID F. JAMES. J. Bact. **39:** 297 (Mar. '40). Natural distribution of bacteria in films prepared for microscopic counting is not taken into consideration when square films are used or random observations made. Preparation of circular films and microscopic sampling along 2 diameters at right angles takes into account concentric distribution of bacteria and minimizes effect of abnormal distribution. Production of reliable counts necessitated use of suitable fixative, composed of "Difco" dehydrated milk, 10%, alum, 0.5% and formaldehyde, 0.4%. Attempt was made to learn minimal number of fields permissible to sample for obtaining estimates substantiated by counting larger number of fields. Relation between ave. no. of bacteria per field and number of fields is discussed.—*Ralph E. Noble.*

**Antigenic Relationships of the Coliform Bacteria.** C. A. STUART, MURIEL BAKER, ALICE ZIMMERMAN, CAROLYN BROWN AND C. M. STONE. J. Bact. **40:** 101 (July '40). Serological findings

appear to confirm biochem. grouping previously proposed for coliform organisms. Evidence of antigenic differences among 3 groups. *Aerobacter* (*Aero.*) group is greatly diversified, with but few antigenic fractions, while *Escherichia* (*Esch.*) is less diversified, with numerous ones. Intermediates appear more diversified than *Aero.* but less than *Esch.* Suggested that organisms now known as coliform, be placed in a single genus, presumably *Esch.*, with three species—*aerogenes*, *freundii* and *coli*. Antigenically, paracolon group of bacteria appears less closely related to typical *Esch.* than are 3 sections of typical coliforms to each other. In general, fair correlation exists between agglutination and fermentation reactions but insufficient to permit classification of paracolon group on basis of their biochem. reactions. Use of term "paracolon" should be limited to papillae-forming, anaerogenic and non-lactose-fermenting *Esch.*-like organisms associated with gastro-intestinal disturbances; and its use should be discontinued as soon as it is possible to apply more descriptive terms to various aberrant coliforms comprising it. Review of literature and present work shows that variants isolated from single cultures and differing biochemically from parent may be antigenically identical with, related to, or entirely different from parent culture.—*Ralph E. Noble.*

**New Enrichment Methods for the Cultivation of Bacterium Coli and Fecal Streptococci in Water Samples.** T. FOLPMERS. J. Microbiol. Serol. **6:** 22 ('39–'40). For selective cultivation of *Esch. coli* incubate at 45° in filled glass-stoppered bottles 5 parts of water to be examined and 1 part each of following broths: (1) 30 grams Bacto tryptone, 4 grams $K_2HPO_4$ and 15 grams NaCl in 1800 ml. distilled $H_2O$ are adjusted to pH 6.5, sterilized 1 hr. in a Koch sterilizer and treated with 50 ml. sterile 20% NaCOOH and 10 ml. sterile 0.1% aq. crystal violet and sterile distilled $H_2O$ to make 2000 ml. Test for indole and gas production after 24 and 48 hr. (2) Solution of 12.5 grams of $NH_4$ lactate, 12.5 grams *d*-glutamic acid and 7.5 grams $K_2HPO_4$ in 350 ml. of hot tap $H_2O$. Two media are recommended for selective cultivation of fecal streptococci: (1) Solution of 20 grams peptone (Poulenc, Difco or Bacto), 20 grams lactose and 10 grams NaCl in 1000 ml. hot distilled $H_2O$ is adjusted to pH 7.0 and sterilized in Koch sterilizer for 1 hr. Twenty ml. sterile neutral litmus solution is added. 1 part of water to be tested is incubated with 1 part of broth in deep layers at 45°. Deposit is examined for strep. after 24 and 48 hr. (2) Solution of 20 grams peptone, 20 grams caffeine, 6 grams Liebig's extract and 10 grams NaCl in 1000 ml. hot distilled $H_2O$ is adjusted to pH 7.0, sterilized 1 hr. in Koch sterilizer and 10 ml. of sterile 20% glucose solution added. Equal part of water to be examined is added and incubated at 37° in completely filled glass-stoppered bottles.—*C. A.*

**Relationship of $CO_2$ Utilization to Succinic Acid Formation by the Coliform Bacteria.** H. G. WOOD AND C. H. WERKMAN. (*Published in abstract only.*) J. Bact. **40:** 158 (July '40). Inhibition of fixation of $CO_2$ by NaF in propionic acid fermentation has been shown to cause a mole-equivalent decrease in formation of succinic acid (s.a.). Interpreted as additional evidence that s.a. may be formed by union of 1- and 3-carbon compounds. Formation of s.a. from different substrates by coliform bacteria is not uniformly inhibited by NaF. Yield of s.a. from glycerol and citrate is not influenced by presence of fluoride. Implication is that s.a. may be formed without utilization of $CO_2$ in these fermentations. In case of galactose and pyruvic acid fermentation, s.a. formation is inhibited by fluoride. Probably utilization of $CO_2$ in these fermentations.—*Ralph E. Noble.*

**Suppression of Spore Formers in Coliform Isolation.** E. R. HUPP. (*Published in abstract only.*) J. Bact. **39:** 631 (May '40). Spore-forming gas formers which produce many pseudo-positive presumptives in routine exam. were eliminated in enrichments by use of 0.01% sodium lauryl sulfate in broth. By using tryptose instead of ordinary

peptone in lactose (lact.) broth, weaker coliforms seem to be rejuvenated. At least a higher percentage of confirmations were obtained with former. Results from using tryptose and sodium lauryl sulfate in lact. broth indicate superior medium for bacterial analysis. Parallel plantings in 796 enrichments of standard lact. and tryptose lauryl broths gave 463 gas formers in former, none of which confirmed, and no gas formers in latter. Parallel plantings in the two media with a suitable dilution of raw water representing 500 enrichments produced 220 coliforms in lact. broth and 251 in tryptose lauryl broth.—*Ralph E. Noble.*

**Incubation at 44°C. as a Means of Indicating the Presence of *B. coli*.** R. Ferramola and J. J. Monteverde. Bol. Obras Sanit. Nacion. (Argentina) **2:** 248 ('39). Report of continuation of previous work on use of MacConkey broth with incubation at 44°C. for differentiation of *Esch. coli* from other coliform organisms in sewage. Tables of data given, showing classification of 400 strains of coliform bacteria isolated from sewage of Buenos Aires. Strains classified in 7 groups according to methyl red and Voges-Proskauer tests and according to ability to produce indol and utilize citrate. 97.8% of strains classified as typical *Esch. coli* gave acid and gas after incubation in MacConkey broth for 24–48 hr. at 44°C.; 98.9% of strains other than *Esch. coli* (*i.e.* intermediate coliform organisms, *Aer. aerogenes* and *Aer. cloacae*) gave negative results in MacConkey broth. Results indicate following distribution of coliform bacteria in sewage of Buenos Aires: typical fecal *Esch. coli*, Type I, 56.0%; typical fecal *Esch. coli*, Type II (unable to produce indol), 1.23%; *Esch. coli* intermediates, 25%; *Aer. aerogenes*, Type I (unable to produce indol), 15.50%; *Aer. aerogenes*, Type II (producing indol), 1.25%; and *Aer. cloacae*, 0.50%. Pointed out that in sewage predominating type of *Aer. aerogenes* is Type I (unable to produce indol), whereas in water Type II predominates. May be that latter type survives more easily in water; its presence and relative abundance would therefore indicate interval which had probably elapsed after contamination with sewage had occurred.—*W. P. R.*

**Use of Sodium Lauryl Sulfate Lactose Tryptose Broth as a Primary Medium for Detection of Coliform Group.** W. L. Mallmann and C. W. Darby. (*Published in abstract only.*) J. Bact. **40:** 143 (July '40). Using gram-pos. and neg. organisms, bacteriostatic titres were determined for Aerosol, M.A., Nacconol N.R.S.F., Duponol paste W.A., Santamerse No. 1 and Igepon T. Nacconol N.R.F.S. and Duponol paste W.A. inhibited streptococci and *Bacillus megatherium* in dilution of 1:30,000. Aerosol M.A. and Igepon T. showed no inhibitory effect on gram-pos. organisms. Action was not due to surface depressant effect as all of wetting agents had approx. same value. By growth curves, found that in concentration of 1:10,000, neither Duponol paste W.A. or Nacconol N.R.F.S. inhibited coliform organisms. Duponol paste W.A. was added to tryptose lactose broth reported by authors (Jour. A.W.W.A. **31:** 689 ('39)) in concentration 1:10,000. Medium, when used in parallel with standard lactose broth on tap waters, showed that in standard broth, 238 primary tubes were pos. but only 4 confirmed, while only 3 primary tubes in lauryl sulfate tryptose broth were pos., all confirming. On raw waters, pos. primary tubes in latter medium, failing to confirm, were found by other technic to contain coliform organisms.—*Ralph E. Noble.*

**The Incidence of Slow Lactose Fermenting Organisms in Water Supplies.** Ira G. Collins. (*Published in abstract only.*) J. Bact. **40:** 335 (Aug. '40). 125 slow lactose fermenting organisms were isolated from 3,106 water samples received from Oct. 9, '39 to Jan. 9, '40. Organisms were obtained by inoculating 1 ml. of each sample into standard lactose broth and incubating at 37°C. Tubes were held 14 days unless fermentation appeared before. EMB plates were used for isolation. 1,513 samples showed gas sometime during period. 125 slow lactose fermenting organisms

were isolated. Of these, 28 were from raw waters, 5 from coagulation basins, 27 from filter effluents, 34 from finished waters, 21 from public school wells, and 10 from private wells. Tests of organisms were made with standard sugar mediums (lactose, glucose, and sucrose), indole, methyl-red, Voges-Proskauer, sodium citrate, uric acid, gelatin and hydrogen sulfide. From 125 strains there were positive: 24 indol, 48 methyl-red, 96 Voges-Proskauer, 109 sodium citrate, and 104 uric acid. 45 liquefied gelatin. In several cases slow lactose fermenters were isolated from same source at different times.—*Ralph E. Noble.*

**Comparative Study of Standard Lactose Broth and Modified Eijkman Medium for Isolation of *Escherichia Coli* from Nut Meats.** Morris Ostrolenk and Albert C. Hunter. Food Research **5:** 141 ('40). Eijkman method for examination of pecans failed to enhance recovery of *Esch. coli.* Although Eijkman technique surpassed standard lactose broth in limiting development of *Aerobacter* and *Citrobacter*, it reduced development of *Escherichia.*—*C. A.*

**Comparative Studies of Lactose Broth and Brilliant Green Bile, at Cambridge, Mass.** Fred E. Smith. J.N.E.W.W.A. **54:** 16 (Mar. '40). Comparative tests made with lactose broth and brilliant green bile (BGB) using latter as both enrichment and confirmatory media. Found that BGB can be satisfactorily used on raw water as direct inoculum, but is slightly inhibitive in this form for coagulated, filtered, and chlorinated waters. Ave. analysis Cambridge water, raw and effluent respectively: color, 43, 4; alk. 19, 23; pH 6.8, 8.7; bacteria per ml. on agar 49, 1; indicated coliforms per 100 ml. 66.0, 0.11. Results of parallel planting tests on 200 raw water samples gave 121 positive coliforms (60.5%) with lactose broth, 132 positive with BGB (66%). On coagulated water (20 samples) percentages of positives were, lactose broth, 33.5%, BGB, 24%; on 113 filtered water samples, broth, 2.7%, BGB, 0%; and on 150 plant effluent samples, broth, 2.7% and BGB, 0%. When BGB used as confirmatory media high degree of correlation with standard methods completed test; found 88.7% confirmations with BGB against 87.2% standard methods on raw water samples, 51.3% and 54.6% respectively on coagulated water tests; 64% and 64% on filtered; and 31.3% and 31.3% on effluent samples.—*Martin E. Flentje.*

**The Problem of the 48-Hour Gas Former and Its Significance From a Practical Viewpoint.** W. B. Schworm. (*Published in abstract only.*) J. Bact. **40:** 336 (Aug. '40). Problem of significance of 48-hr. gas former on lactose peptone broth has engaged attention of water bacteriologists for a no. of yr. Mainly, organisms are spore-forming anaerobes, failing on Endo or eosin methylene blue, highly resistant to chlorine, bromine, iodine and silver nitrate. Tolerance of organisms for these bactericides was established by tests in St. Louis Water Div. Labs. Present Treas. Dept. standards for drinking water for interstate carriers does not include bacteria other than members of colon group, as tested by *Standard Methods*, but public health officials are inclined to look upon water supply with suspicion in outbreaks of gastro-enterititis. Whether attitude is justified or not is debatable, since no clinical evidence has yet come to light establishing a connection between chlorine-resistant 48-hr. lactose fermenters and miscellaneous gastro-intestinal disturbances. Program of determinative bacteriological collaboration of a number of water works labs. is suggested as a means of establishing this knowledge and ending present dilemma.—*Ralph E. Noble.*

**The Presence of Coliform Bacteria in Chlorinated Water.** Geo. W. Freiberg. W. W. & Sew. **87:** 65 (Feb. '40). At brewery water treatment plant, coliform organisms were thoroughly removed in summer; but at low temps. they persisted in chlorinated filtered water, except when water was caustic. Use of excess lime treatment was adopted to help kill organisms. Thorough experiments were undertaken, procedures for

which are described. These showed excellent bactericidal action on water which had been kept caustic long enough to attenuate organisms. Seems necessary to operate treatment plants in such a way as to produce negative presumptive tests. Short-circuiting and other local conditions may necessitate adjustment of causticity and chlorine dosage. —*H. E. Hudson Jr.*

**A Study of Floc-Forming Organisms in Chlorinated Water Supplies.** A. M. Buswell and Max Suter. J. Bact. **39:** 583 (May '40). A water supply problem, little touched upon, is growth of unsightly flocs of saprophytic bacteria in chlorinated water of districts where it is high in alkalinity, hard, low in chlorides and sulfates, and contains unoxidized Fe, S, $NH_3$ and other substances. Usually growth is heaviest in July and Aug. Microscopically, sludge is chiefly composed of zoögloeal masses comprised of capsulated small cocci or short rods. Formerly, it was thought *Crenothrix polyspora* was main cause of such floc, but this organism does not spread beyond aerator. Organisms found in distribution system are, *Leptothrix ochracea* and *Gallionella ferruginea*, while *Siderocapsa* and *Sideroccoccus* are most common. Fe bacteria form only small part of floc organisms, due to less than 0.1 p.p.m. Fe content. $NH_3$ is a source of energy which small capsulated cocci or short rods convert to nitrites. Resistance of floc to disinfectants was tested by using sludge containing 100.0 p.p.m. total solids mixed with different disinfectants to give final concn. of 0.5 to 50.0 p.p.m. Organisms survived 48 hr. at room temp. in 2.0 p.p.m. $AgNO_3$, 15.0 p.p.m. $HgCl_2$, and 30.0 p.p.m. $Cl_2$. $CuSO_4$ and $K_2Cr_2O_7$ caused some reduction at 50.0 p.p.m. $BaCl_2$, $NiSO_4$, $As_2O_3$, KCN and sod. pentachlorphenate exerted no influence at latter concn. Oligodynamic action of metallic Cu, Pb, Hg and Ag is considerable and caused free zones of growth around metal. Reduced growth was found around As, Cr, and Al. No effect exerted by Zn, Fe, Sn, Co, Pt, Au, Tu, Bi, Ta or Ir. No stimulated growth.—*Ralph E. Noble.*

**Influence of Chlorine, Sodium Hypochlorite and Iodine on the Hydrolytic Action of Pepsin.** Ilie Matei and Elena Cocea. Ann. Sci. Univ. Jassy. (Rumania) **26:** I: 41 ('40). Hydrolytic action of pepsin is inhibited profoundly in presence of chlorine, sodium hypochlorite or iodine in concentrations of 15–65 milli-equivalents per liter of hydrolyzable medium. Slight activation is produced by chlorine and sodium hypochlorite in concentrations of about 8 milli-equivalents. Chlorine, in large concentrations augments; sodium hypochlorite depresses; and iodine has no effect on acidity of medium under given experimental conditions. Purification of natural water supplies with chlorine is probably advisable, since chlorine in small concentrations provokes feeble activation of peptic hydrolysis.—*C. A.*

**The Incidence of *Aerobacter Aerogenes* in Human Fecal Excreta.** R. J. Reedy and J. F. Puncochar. (*Published in abstract only.*) J. Bact. **39: 757** (June '40). 253 samples of human feces obtained from 253 normal adult women were tested for presence of *Aer. aerogenes*. Approximate ratio of aerogenes, or citrate utilizers, to coli was determined. Enrichment in citrate was followed by streaking on E.M.B. Typical colonies were picked and planted into citrate. All organisms were considered to be aerogenes that were gram-neg., non-spore-forming rods, M.R. neg., V.P. pos., citrate pos. and indol pos. Aerogenes was detected in 220 samples, an incidence of 87%.—*Ralph E. Noble.*

**Colon-Aerogenes Organisms in the Natural Waters of the Argentine Republic.** R. Ferramola and A. Gelstein. Bol. Obras Sanit. Nacion. (Argentina) **3:** 159 ('38). Authors discuss interpretation of bact. analyses of water with special reference to significance of *Esch. coli*, *Aer. aerogenes* and coliform intermediates. *Esch. coli* is predominating coliform organism in feces of man, animals and birds. *Aer. aerogenes* occurs in uncontaminated soil, grains and plants, but may be found in relatively small numbers in human feces. Tables, showing distribution of *Aer. aerogenes*

in human feces, and of *Esch. coli* in water, given. Graph, showing correlation between presence of *Esch. coli* in drinking water and mortality due to typhoid, also included. Presence of *Esch. coli* in water is accepted as evidence of recent contamination by fecal matter. Presence of *Aer. aerogenes* may be result of less recent fecal contamination or of contamination by soil. Sources of water containing *Aer. aerogenes* are questionable and should be examined frequently. Results of an investigation of distribution of different species of coliform bacteria in samples of chlorinated and unchlorinated surface and ground waters from different parts of Argentina given in tables. Bacteria classified according to scheme adopted by Ministry of Health, Great Britain; *i.e.*, according to their ability to utilize citrate, produce indol and liquefy gelatin, and according to methyl red and Voges-Proskauer tests. Of 96 strains isolated from unchlorinated surface waters, 58% were *Esch. coli*; 22%, intermediates; and 20%, *Aer. aerogenes*. Corresponding figures for 129 strains isolated from chlorinated surface waters were 42%, 32%, and 26%. Of 297 strains from unchlorinated ground waters, 10% were *Esch. coli*; 52%, intermediates; and 38%, *Aer. aerogenes*; whereas 50% of the 36 strains obtained from chlorinated ground waters were intermediates and the remainder *Aer. aerogenes*. Discussing results, authors point out that, in interpreting san. signif. of *Esch. coli* in water, geographical configuration, population and climate must be taken into consideration. In thickly populated areas, organisms must be regarded as evidence of pollution by human fecal matter; in sparsely populated districts, organisms are probably of animal or avian origin, and, therefore, probably of less san. signif.—*P. H. E. A.*

**A Biochemical Study of the Fermentation of Rare Sugars by Members of the Colon and Aerogenes Groups of Bacteria.** *III.* d-*Fucose*. J. THOMAS FIELD AND CHARLES F. POE. J. Biol. Chem. **132:** 473 ('40). Some 42 organisms of *Escherchia* and *Aerobacter* groups of bacteria were used in determining gas formation in media containing fucose. Gas production took place very slowly and in 96 hr., colon cultures averaged 8% and aerogenes cultures 11% of gas. pH values with both groups of organisms in media containing 0.1% fucose reached min. in about 7 hr. (5 to 5.25) and then gradually increased to max. of about 8.7. Min. was considerably lower (4.6) with 0.5 and 1.0% fucose, and acidity which developed killed organisms in about 30 hr. After 72 hr. incubation, aerogenes group produced on ave. about twice as much lactic acid and 4 times as much succinic acid as colon group and ratio for volatile to nonvolatile acids and for acetic to succinic acid were more than twice as large as for colon group. All members of aerogenes group also formed acetylmethylcarbinol. Colon group produced more gases and alcohol. —*C. A.*

**An Ecological Study of the Coliform Bacteria.** A. M. GRIFFIN AND C. A. STUART. J. Bact. **40:** 83 (July '40). Distribution of 6,577 strains of coliform organisms in milk, water, soil, grains, and feces suggests that *Aerobacter* and intermediates constitute normal coliform flora of non-fecal materials, while *Escherichia* are normal to feces. Suggested that occurrence of groups outside normal habitats is adventitious. Stability of indol test as compared with citrate utilization renders it more suitable for taxonomic purposes. Most aberrant lactose fermenters appear to be of questionable san. signif. Some question as to advisability of continuing distinction between *Aer. aerogenes* and *Aer. cloacae*.—*Ralph E. Noble.*

**Sensitivity of *Escherichia Coli* to Cold-Shock During the Logarithmic Growth Phase.** C. P. HEGARTY AND OWEN B. WEEKS. J. Bact. **39:** 475 (May '40). Observations of Sherman and Albus that young cells of *Esch. coli* are susceptible to initial cold-shock from 37°C. to 0°C. have been confirmed. Sensitivity to cold-shock extends throughout entire logarithmic growth phase. Mature cells not affected by initial cold-shock or prolonged holding at 0°C. During period between lag and very early part

of log. growth, sudden cold-shock has but slight initial effect. Upon holding at 0°C., subsequent to stationary period, there is marked increase in no. of cold-susceptible cells. Indicates phenomenon is concerned with changes within cell, rather than to physical effect alone. Frequent plate counts have shown growth curve to progress in a step-wise manner, due to simultaneous division of many cells. Sensitivity of *Esch. coli* to cold seems to be related in some manner to cell division and to changes within individual cell. 19 refs.—*Ralph E. Noble.*

**Limitations of Euglenidae as Polluted Water Indicators.** JAMES B. LACKEY AND RUSSELL S. SMITH. Pub. Health Repts., **55**: 7: 268 (Feb. 16, '40). From Apr. 37 to Nov. '38, 844 plankton samples were collected from 18 stations on the Scioto R. and its tributary creeks. Each station was sampled weekly for majority of period. Half the samples were preserved with formalin while remainder were examined fresh and usually after concentration by centrifugation of a 100 ml. volume. Microscopic examinations were made by Lackey. 48 species of green Euglenidae occurred, divided among 5 genera. Since specific determination was not possible in all cases, 1 group of *Euglena* and 1 of *Trachelomonas* were simply designated as "Species" although they may have been a complex of integrading species.

5 figures and 1 table illustrate and summarize data. One figure is a diagrammatic presentation of the shapes of 11 integrading forms which are all referred to the species *Trachelomonas urceolata*. Two others give the 19-month ave. of green Euglenidae at each station and the seasonal distribution 23 mi. below the Columbus sewer outfall. Station by station ave. of *Euglena viridis* and *Trachelomonas urceolata* are also given graphically.

Euglenidae showed marked seasonal distribution and as a *group* did not appear to be good index of pollution. They occurred in zone of recovery rather than in areas of noticeable pollution. There was, therefore, no good correlation with other measures of pollution, i.e. with suspended solids, 5-day B.O.D., pH and 37° agar count. There is a digest of past observations by Lackey and others which points to unreliability of *group* as a pollution index.

Authors believe *Euglena viridis* only species from Scioto R. study valuable as an indicator, and even this form may be abundant in a zone where pollution is hard to detect by other means. Experimental studies of the authors and their survey of literature leads them to conclude that the following species tend to occur in abundance in sewage-polluted water: *Cryptoglena pigra, Euglena fusca, E. Gracilis, E. oxyuris, E. pisciformis, E. polymorpha, E. viridis, Lepocinclis texta, Phacus pyrum, Trachelomonas crebea,* and *T. urceolata.* Authors express opinion that abundance of these organisms *may* mean pollution (probably indicating organic substances or end products of decomposition) and that these forms should not be regarded as pollution indicators unless supported by other evidence.—*Theodore A. Olson.*

**Iron Bacteria, the Fauna and Flora of Biological Films.** G. BATTA. Rev. Univ. Min. (Fr.) **15**: 568 ('39). (*Epuration des Eaux I*). Iron bacteria develop mainly in water containing much iron and manganese and have property of precipitating iron and, in most cases, manganese. Characteristics of types occurring most frequently (*Crenothrix polyspora, Leptothrix ochracea,* and *Gallionella ferruginea*) and difficulties caused by their presence in water supplies are described. These bacteria are not dangerous to health but they may cause incrustations and corrosion in pipes and clogging of filters, and may impart undesirable color and taste to the water. Corrosion of pipes may also be caused by anaerobic organisms, such as *Vibrio desulfuricans,* which are present in soils. Theories of process of corrosion by iron and other bacteria are discussed. Author then deals with other micro-organisms of importance in operation of water works. Organisms which form film on sand of filters play important part in purification of water. List is given of organisms found in film on filters at Waelhem near Antwerp. Algae and protozoa may develop in

settling tanks and in some cases cause odors in water. Disinfection of distribution system with calcium hypochlorite is generally expensive; silver nitrate or copper sulfate in very small quantities can be used to destroy growths.—*W. P. R.*

**Anaerobic Flora of the Human Intestine. *Acuformis Perennis*, New Species.** A. R. PRÉVOT. Compt. Rend. Soc. Biol. (U.S.S.R.) **133:** 574 ('40). Organism is rod-shaped, strictly anaerobic, non-motile, spore-forming, gram-positive, non-pathogenic and grows well between pH 5.8 and 8.2. Clots milk and does not liquefy gelatin. Ferments glucose, fructose, maltose, galactose, sucrose, arabinose and lactose, forming propionic, valeric and lactic acids and gas containing hydrogen. Forms volatile amines but no indole, hydrogen sulfide, ketones, ethyl alcohol, phenols, acetylmethylcarbinol, toxin or hemolysin.—*C. A.*

**Some Factors Which Influence Oxygen Consumption by Bacteria in Water.** CLAUDE E. ZOBELL. Biol. Bul. **78:** 388 ('40). Respiration of lake bacteria increases with temp.; the $Q_{10}$ from 8 to 25° being 2.1. Mixed microflora from Lake Mendota, Wis., found to consume from 21 to $43 \times 10^{-12}$ mg. oxygen per cell per hr. at 25°. Rate of respiration is independent of oxygen concentration within examined range 0.31 to 26.48 p.p.m. Below 0.3 p.p.m., respiration decreases rapidly. About ¼ of organic matter of Lake Mendota water is oxidizable. Organic content of bottom deposits is less oxidizable than that of water, and remains of plankton organisms are more readily oxidized by bacteria than dissolved organic matter occurring in lake water. Effect of adding organic compounds was studied. —*C. A.*

**Bacteriological Survey of Indoor Swimming Pools in Province of Quebec.** RENÉ CYR. Can. Engr. **78:** 9: 13 (Sept. '40). Tabulated results covering period '30–'40 show that bacteriological quality of swimming pool water is improving as result of control exercised by municipal and provincial officials. Samples containing over 200 bacteria per ml. have been reduced from around 46% during early part of decade to 26% during latter part; and percentage containing coliform organisms in 10 ml. from 26% to 5%. Standards require that not more than 15% of samples shall contain more than 200 bacteria per ml. (37°C.) or show positive coliform test in 10 ml. Of 10 pools submitting samples in '39–'40, 7 conformed to standard for total count and 9 to standard for coliform organisms. —*R. E. Thompson.*

## QUALITY

**Standards and Regulations for Quality of Water in Bathing Places.** A. E. BERRY AND A. V. DELAPORTE. Can. Engr. **78:** 8: 5 (Aug. '40). General similarity in standards in U. S. and Canada. Unfortunately, no rule regarding interpretation of analytical results recognized as applicable under all conditions. More variation occurs in standards for outdoor bathing areas than in those for artificial pools operated under adequate supervision. Standard recommended by Joint Committee on Bathing Places (Conference, State San. Engrs. and A. P. H. A.), *i.e.*, that not more than 15% of samples covering any considerable period of time shall contain more than 200 bacteria per ml. or show positive confirmed test (coliform bacteria) in any of five 10-ml. portions of water when pool is in use, is reasonable for pools but natural outdoor bathing places will not comply with such rigid standard. Following coliform bacteria standard for outdoor bathing areas has been given some prominence: Class A, 0–50 per 100 ml.; Class B, 51–500; Class C, 501–1,000; Class D, over 1,000—Class A being regarded as good, D as very poor, and 2 intermediate classes as doubtful and poor. It can be assumed that 1,000 coliform bacteria per 100 ml. is reasonable dividing line, below this figure being acceptable under average condi-

tions and above unfavorable. Source of pollution, of course, should always be considered. Only practical measures in regard to natural bathing areas are selection of site free from gross pollution, and treatment of polluting material at point of discharge. Usual method of treatment for pool water is filtration through pressure units, using alum as coagulant, and chlorination. Turnover should be twice per day. No limit to no. of times water can be filtered and returned to pool if alkalinity is maintained by addition of soda ash, etc. Joint Committee recommends maintenance of 0.4–0.6 p.p.m. residual chlorine in pool water at all times. Compulsory foot baths containing 3,000–6,000 p.p.m. chlorine are used to control fungus foot. Mats and other places where the organisms may grow are to be avoided and thorough scrubbing of floors and surroundings with chlorine wash is good precaution.—*R. E. Thompson.*

**The Modern Conception of the Bacterial Quality of Water.** JOHN R. BAYLIS. (*Published in abstract only.*) J. Bact. **40:** 146 (July '40). Author recommends adoption of bacterial standard more rigorous than that known as "Treasury Dept. Standard." Complete elimination of lactose-fermenting organisms is suggested. More than 1.0% of 10-ml. tubes or more than 10.0% of 100-ml. tubes [positive] within a month is unsatisfactory. Total count at 37°C. should not be more than 2 per ml., nor should more than 50% of pos. presumptives confirm. Sewage polluted water should carry 0.5–1.0 p.p.m. residual chlorine for 2 hr. regardless of bacterial purity of water. Chlorine-resistant organisms should be removed regardless of amount of chlorine necessary.—*Ralph E. Noble.*

**Practical Aspects of the Bacteriological Control of Public Water Supplies.** W. SCOTT JOHNSON. (*Published in abstract only.*) J. Bact. **40:** 336 (Aug. '40). Modern practices dictate that safety of pub. water supplies shall depend upon 4 factors: (1) condition of raw water, (2) design and construction of treatment plant, (3) operation and maintenance of treatment plant, and (4) lab. control, including water bacteriology. Long been recognized that 2 factors are of vital importance in practical use of bact. data in control of public water supplies—analyses must be made in accordance with (1) uniform standard procedure and (2) a standard of allowable frequency of coliform occurrence. Although, from time to time, *Standard Methods* and the Treas. Standard have been changed and improved, and future developments are desired, they have constituted a most useful, valuable and practical tool for those concerned with water safety for drinking purposes. Investigators have, from time to time, questioned efficiency of standards due to outbreaks of intestinal disturbances which had indications of being water-borne, but in most cases could not be definitely proven so. Among other criticisms, it has been maintained: (1) that *Standard Methods* was not sufficiently sensitive; (2) that more time should be allowed for gas formation; and (3) larger portions of water should be analyzed. Treas. standards are criticized because of allowing too large percentages of positive 10 ml. quantities, and due to elimination of plate count.—*Ralph E. Noble.*

**Bacteriological Standards for Public Water Supplies and the Treatment Required.** H. O. HARTUNG. (*Published in abstract only.*) J. Bact. **40:** 335 (Aug. '40). Some investigators and pub. health authorities suggest present bact. criterion of safe water supply (Treas. Standard—1 coliform organism per 100 ml.) might not be sufficient safeguard against all water-borne disease. Indicate that mild and semi-mild intestinal disturbances, which become epidemic, could be water-borne, even though water in question satisfied Treas. Standards. While epidemiological data do not support such opinion, argued that drinking water should be above mild suspicion, believing this possible only when water is bacteria-free. To produce bacteria-free water is possible but not

without difficulties and considerable expense. Missouri R. water may require as much as 20 p.p.m. chlorine, if Cl contact period is about 24 hr.; 1 to 2 p.p.m. Cl will kill coliform organisms, allowing water to meet present Standard. If ammonia-chlorine treatment is used, even 25 p.p.m. Cl will not completely sterilize water. pH during treatment also has tremendous influence on bacteria kill. Water plant operator desires to produce water consistent with adequate public health safeguards, but at same time is reluctant to subject community to needless expense.—*Ralph E. Noble.*

**The Necessity for Bacteriological and Hygienic Examination of Drinking Water.** W. SAUER. Off. GesundhDienst. (Ger.) **4:** 423 ('38); Gas u. Wasser. (Ger.) **81:** 896 ('38). Police regulations enforcing regular bact. exam. of central water supplies are lacking in most of Prussia. Occasional chem. exams. should be undertaken as well as frequent bact. exams. Small communities have for a considerable time had tests carried out for them by chemists. Effort on part of food control authorities to take over bact. control of water supplies but this is not approved of by health authorities. Many deficiencies, of which author quotes examples, were discovered in official inspection of water works in Erfurt, proving that regular bact. and hygienic exam. of water is necessary. Author concludes that bact. investigations must be carried out by medical bacteriologists, and not by chemists or food control authorities.—*W. P. R.*

**The Interpretation of the Results of the Chemical and Bacteriological Examination of Drinking Water.** J. A. MCLACHLAN. Pub. Health (Johannesburg, S. Afr.). **4:** 5: 13 ('39). Results of chem. and bact. examn. of 10 samples of water are tabulated and significance is discussed. High values for free and saline $NH_3$, albuminoid $NH_3$ and O absorbed indicate contamination with organic matter; oxidized N indicates extent to which impurities have been oxidized; high concns. of chloride generally indicate pollution from animal sources as does also presence of *Esch. coli*. Opinion on state of water must in every case be based on a consideration of analysis as a whole and not on any one test.—*W. P. R.*

**Importance of Chemical Analysis in Determining the Potability of Water, With Reference to Some Peruvian Waters.** FRANZ KOHN. Bol. Soc. Quim. (Peru). **6:** 2: 16 ('40). Chem. criteria of potable waters are discussed. In Peru, criteria do not in all respects parallel those set up for European countries, where limits are imposed on Cl, $NO_2$ and $NO_3$. In some parts of Peru these anions occur in considerable amts. in mineral form, and their presence in a water does not raise suspicion of contamination unless confirmed by bact. evidence and presence of $P_2O_5$. Analyses of 60 waters are reported.—*C. A.*

## WELLS AND GROUND WATER

**The Source of Water Derived from Wells.** CHARLES V. THEIS. Civ. Eng. **10:** 277 (May '40). Essential factors controlling action of aquifer appear to be: (1) distance to and character of recharge; (2) distance to locality of natural discharge; and (3) character of cone of depression in given aquifer. Recharge to aquifer may result from penetration of rainfall through soil to water table, or by seepage from streams or other bodies of surface water, or by movement vertically or laterally from another ground water body. Rate of recharge is governed by rate at which water is made available by precipitation or flow of streams, or by rate at which water can move vertically downward through soil to water table and thus escape evaporation. Normal recharge of aquifer is sometimes assumed to be measure of possible yield of aquifer to wells. Ground water flows through an aquifer proportional to pressure gradient in water. Two fundamental physical properties of any aquifer which largely control movement of water through it. First is ease with which it transmits water and other is amount of water that will be released from storage when head

in aquifer falls. On simplifying assumption that removal of water is analogous to removal of heat from a metal plate, an equation for the drawdown caused by pumping a well may be derived. This formula for cone of depression in the ideal homogeneous and isotropic aquifer assumed is:

$$r = \frac{114.6F}{T} \int_z^\infty (e^{-u}/u)du$$

in which $V$ = drawdown at any point in ft.; $F$ = rate of discharge of the well in g.p.m.; $T$ = coefficient of transmissibility; $z = 1.87r^2s/Tt$; $r$ = distance between pumped well and point of observation in ft.; $s$ = coefficient of storage; $t$ = time well has been discharging in days; $u$ = a dimensionless quantity varying between limits given. Res. from which well takes water is almost as closely circumscribed by time as it would be by any material boundary, and until sufficient time has elapsed for cone to reach areas of natural discharge and rejected recharge, new equilibrium on aquifer cannot be established. Importance of time effect varies with characteristics of aquifer and distance of well to areas of recharge and natural discharge. After cone of depression reaches areas of rejected recharge or natural discharge, it is modified by effects of adding water in former or preventing it from escaping in latter. In summing up: (1) All water discharged by wells is balanced by loss of water elsewhere. (2) This loss is always to some extent and in many cases largely from storage in aquifer. (3) After sufficient time has elapsed for cone to reach area of recharge, further discharge by wells will be made up, at least in part, by increase in recharge, if previously there has been rejected recharge. (4) After sufficient time has elapsed for cone to reach areas of natural discharge, further discharge by wells will be made up in part by a diminution in natural discharge. (5) In most artesian aquifers little of water is taken from storage. (6) Ideal development of any aquifer from standpoint of max. utilization of supply would follow these points: (a) Pumps should be placed as close as economically possible to areas of rejected recharge or natural discharge, where ground water is being lost by evaporation or transpiration, or where surface water fed by, or rejected by, ground water cannot be used. (b) In areas remote from zones of natural discharge or rejected recharge, pumps should be spaced as uniformly as possible throughout available area. (c) Amount of pumping in any one locality would be limited. In localities developing water from nonartesian aquifers and remote from areas of rejected recharge or natural discharge, condition of equilibrium connoted by concept of perennial safe yield may never be reached in predictable future and water used may all be taken from storage.—*H. E. Babbitt.*

**Analysis of Legal Concepts of Subflow and Percolating Waters.** C. F. Tolman and Amy C. Stipp. Proc. A.S.C.E. **65:** 1687 (Dec. '39). (*See* Jour. A.W.W.A. **32:** 891 ('40). *Discussion: Ibid.* **66:** 779 (Apr. '40). Samuel C. Wiel: Legal hospitality to data of hydrology indicates more to come as hydrology is patiently presented. In newness it is natural that there are conflicts among hydrologists. New science's debates slow its reception. Perhaps most important moderator on rate of reception of hydrology into law is fact that what takes place in courts about water never occurs in abstract as in classroom or office, but is part of drama of opposing sides. Would it be agreeable to hydrologists to have a judge conjecture on a locality's hydrology by judicial notice without any witness testifying? Failure to produce evidence reflects less on court than on litigants. If hydrology's entry into judicial decisions is slow or sometimes imperfect, it is fair to assume that its acceleration waits on practical considerations. Disregard of indeterminateness of subject leads to embracing some form of legal separation of rights by process of dividing up water resources into independent parts like land lots. One form thinks of dividing water-body into detached masses by vertically projected land-boundary lines. Law is increasingly coming to rely on how a situation appeals to good sense. In-

stances in which public use meets legal hindrance usually find the public use insisting upon proceeding with its work while leaving question of amount of damage unheard and untried. HYDE FORBES: Writer has found attorneys quick to understand and appreciate geologic conditions and hydrologic phenomena and able to express these conditions in clear and concise language necessary in legal proceedings. Engrs. and geologists who, through experience, have gained a knowledge of groundwater hydrology can clarify seeming misconceptions and inconsistencies if, when controversies arise, they will recommend to their clients and attorneys that they be allowed to make investigations jointly with representatives of other party to controversy to the end that questions of fact will be settled through stipulation. Lawyers and courts will then have no trouble in applying law to facts. RONALD B. HARRIS: Writer recognizes justification of criticism that certain court decisions have expressed erroneous concepts of nature of ground water. Courts are finders of facts which they cull from evidence presented to them. Place to correct misconceptions of nature and characteristics of ground water is at source of learning, and that is, at universities. *Ibid.* **66:** 1020 (May, '40). EDWARD F. TREADWELL: Writer disagrees entirely with criticisms made of decisions in question. Such a paper would be of much more value if it had been prepared in cooperation with a lawyer. Principle of geology can no more be applied than can a principle of law, until facts are determined. Court does not violate principle in either case because it refuses to apply it on ground that necessary facts have not been established. Writer is entirely unable to understand wherein any of these cases violates any positive rule of hydrology, even if he were to admit correctness of all rules stated by authors. O. E. MEINZER: In showing clearly the confused and erroneous character of many legal concepts relating to ground waters, and in substantiating previous discussions of this important subject, authors have rendered a valuable service. Erroneous legal concepts regarding ground waters were first presented to courts at a time when true scientific concepts had not yet been well developed. Authors do not discuss subject "Effects of Subflow of Diversions from Influent and Effluent Surface Streams". In most cases it is user of ground water who has been on defensive and case has generally been decided in favor of the user of stream water. M. R. LEWIS: In case of subsurface stream whose banks are permeable, shape of contour close to subsurface bank in less pervious material will be more, rather than less, pronounced influent. To enable courts to consider general applications of groundwater hydrology and law when they are called upon to decide specific cases, it would be well if appropriate state or federal agencies were permitted and empowered to appear as "friends of the court" in order to present such general applications. BAYARD F. SNOW: No doubt that many legal decisions have been based upon insufficient or misleading engineering assistance and have in turn led to lines of questioning which have made it difficult, if not impossible, for competent witnesses to present the truth to the court. There is, perhaps, a danger that in simplified examples given, a reader, particularly if looking for an explanation to fit his conditions, may overlook certain fundamental facts. Law has divided available ground water into two classifications and has differentiated between rights which may be acquired in accordance with that classification; but if cited definition is to govern, rights to "percolating waters" are of no practical value. Although all underground waters are presumed to be percolating, and to take them out of the rule with respect to such waters, existence and course of a permanent channel must be clearly shown, distinction is one of degree and is not absolute. Foregoing can be illustrated by reference to court decisions in at least 8 states. Another result of legal misunderstandings of groundwater hydrology and of legal precedents is illustrated in decisions in 4 states. Although a few decisions have recognized that all underground waters are percolating, in gene-

ral, the courts have distinguished between percolating waters and underground streams. *Ibid.* **66:** 1130 (June, '40). HAROLD CONKLING: Study of ground water makes desirable revision of definition which author cites: "A stream of water has a defined channel. It has banks and is very distinct from the percolations of subsurface water, which oozes in veins or filters through the earth's strata." Definition describes only surface portion of stream. Entire stream may be many times wider and deeper. Following definition of stream is offered: "A stream is a body of water moving as a whole in an accustomed location and in a definite direction." With information now available, and with its crystallization into conception of stream hereinbefore expressed, would seem probable that, in near future, subsurface water involved in many cases can be treated as a stream by experts and courts, thus removing concepts in previous decisions which author mentions.—*H. E. Babbitt.*

**Local Overdevelopment of Ground Water Supplies, With Special Reference to Conditions at Grand Island, Neb.** L. K. WENZEL. U. S. Geological Survey. Water Supply Paper 836-E ('40). Report describes quantitative studies of relation of pumpage to replenishment of ground water supply in Grand Island, Neb. area. Total pumpage from wells in ground water res. in Pleistocene sand and gravel that underlies Grand Island and vicinity (area about 8 sq. mi.) was about 10,850 acre-ft. per day, or ave. of 9.7 m.g.d., in '37. Of this, however, 9,100 acre-ft. was pumped from about 8 sq. mi. in and very near Grand Island. During 2-yr. period, '36-'37, total of 16,210 acre-ft. of water pumped from this small area. Hydrologic data, including information on pumpage, water level observations, permeability, and specific yield of formation, collected during investigation, presented and analyzed. Pumpage was more than amt. of water reaching area by underflow. Excess draft supplied partly by recharge from precipitation within area, and partly by removal of ground water from storage. Although quantity of water removed from storage represents only small percentage of water pumped, present pumpage is greater than can be perennially supplied under existing hydrologic conditions. Author indicates how problem may be solved.—*Ed.*

**Conservation of Water Through Recharge of the Underground Supply.** A. T. MITCHELSON. Civ. Eng. (Easton, Pa.). **9:** 163 ('39). "Water spreading" defined as practice of diverting surface water from natural channels and spreading it over adjacent porous beds for purpose of augmenting and replenishing ground water supply. Frequently used in Calif. where surface reservoirs are difficult and costly to construct and where there are marked seasonal fluctuations in surface waters. Porous beds used for storage must be close to channel of stream and above level of wells which it is desired to replenish. Detritus cones at mouths of canyons are usually suitable, although control of water in them may be difficult. Methods of spreading water include impounding it in series of small basins formed by dikes often made to follow contour lines. Method is suitable where surface of ground is irregular, but not when water contains large amounts of silt, as this tends to seal surface. Systems of ditches, arranged in various ways, provide second method of spreading water. This method suitable in stream beds, especially where danger of flooding prevents installation of permanent works. Most efficient method is to spread water in thin sheet over land but this is only possible on areas with gentle slopes and with no large gulleys or ridges. Greatest percolation occurs on areas where natural vegetation and soil have not been disturbed. Underground supply may also be replenished by returning surface water to shafts and pits. Some experiments in progress in Utah are described briefly.—*W. P. R.*

# APPLICATION FOR MEMBERSHIP
# IN THE
# AMERICAN WATER WORKS ASSOCIATION

**22 EAST 40th ST., NEW YORK**

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Date:.....

..... hereby make application for
(I or We)

.....
(Active, Junior, Corporate or Associate Membership, or Affiliate)

in the American Water Works Association, and enclose herewith the sum of $....., one year's dues in advance.

Name.....

Company or Department.....

Title or Position.....

Address.....

.....

If application is for Junior Membership, give date of birth.....

If application is for Affiliate, state number of active services in property where employed.....

Nature of business or character of work (for office records).....

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If application is for Corporate or Associate Membership, it must be signed by the person designated to represent the firm or corporation in A.W.W.A. activities.

.....
Signature of Applicant.

Application obtained by:

.....

(over)

## ARTICLE I OF BY-LAWS

Section 3. An Active Member shall be a superintendent, a manager, an official or employee of a municipal or private water works; a civil, mechanical, hydraulic, or sanitary engineer, a chemist, a bacteriologist, or any qualified person engaged or interested in the advancement of knowledge relating to water supplies. (Annual Dues, $10.00.)

Section 4. A Corporate Member shall be a Water Board, Water Commission, Water Department, Water Company or Corporation, National, State or District Board of Health, or other body, corporation or organization engaged or interested in water supply work, and shall be entitled to one representative whose name shall appear on the roll of members, and who shall have all the rights and privileges of an Active Member. This representative may be changed at the convenience and pleasure of the Corporate Member on written notice to the Secretary. (Annual Dues, $15.00.)

Section 5. An Associate Member shall be either a person, firm or corporation engaged in manufacturing or furnishing supplies for the operation, construction, or maintenance of water works. (Annual Dues, $25.00.)

Section 6. A Junior Member shall be an employee of a municipal or private water works; a civil, mechanical, hydraulic, or sanitary engineer, a chemist, a bacteriologist, a student or any otherwise qualified person engaged or interested in the advancement of knowledge relating to water supplies. At the time of his admission he shall be not less than eighteen years of age. His connection with the Association shall cease when he becomes twenty-five years of age, unless he is regularly enrolled as a student in a university or has previously transferred to the grade of Active Member. Junior Members shall receive the Journal and all privileges of Active membership except holding office and voting. (Annual Dues, $5.00.)

Section 7. An Affiliate shall be any person otherwise qualified for Active membership who, at the time of application, is not nor previously has been a member of the Association and who, for acceptable reasons, does not wish to become an Active Member.

No corporation, firm or partnership which otherwise would be entitled to the grades of Associate or Corporate member may hold the grade of Affiliate. No employee of an Associate member may become an Affiliate. No person who is the superintendent, the manager, the chief engineer, the superintendent of filtration, the chief chemist, or the superintendent of distribution in a plant having more than 3,000 active services, is eligible for the grade of Affiliate. Under unusual conditions, exception to the above may be made by action of the Executive Committee if the applicant sets forth fully the reasons for the exception when applying for the Affiliate grade.

Affiliates shall not be entitled to vote upon general Association questions, and not eligible to hold office in the Association, nor in any of its Divisions. They shall be eligible to vote upon Section questions and to hold Section offices except those of Chairman, Vice-Chairman, Secretary (and/or Treasurer). They shall be entitled to all other rights and privileges of Active Members. Affiliates receive the March, June, September and December issues of the Journal each year. (Annual Dues, $4.00.)

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Memberships will be dated as of the beginning of the quarter in which the application is received.

Membership in the Association carries, also with no additional dues, membership in its Local Sections and National Divisions, and includes the Journal, a monthly publication devoted to water works interest. The proceedings of the annual conventions and of the meetings of the Local Sections are published in the Journal, which also contains contributed articles on subjects pertaining to public water supplies.